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Of The
WESTERN CANADA WATER
And
WASTEWATER ASSOCIATION

EDMONTON, ALBERTA
October 4, 5, 6, 1989

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WESTERN CANADA POLLUTION CONTROL ASSOC. W.P.C.F.

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WESTERN CANADA WATER AND WASTEWATER ASSOCIATION

including the

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WESTERN CANADA SECTION

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WESTERN CANADA WATER AND WASTEWATER
ASSOCIATE MEMBER LISTING

GUIDE-LINES FOR ACCEPTANCE TESTING OF NEW WATER-MAINS IN THE CITY OF EDMONTON

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INTRODUCTION

Traditionally, inspectors for the City of Edmonton have been responsible for testing and disinfecting new water-mains. Currently, the consulting engineer and contractor are responsible for pressure testing and disinfecting new water-mains. Bacteriological sampling for total coliforms is performed by the engineer or contractor and the samples are submitted to the Provincial Laboratory of Public Health for analysis. The results are then sent to the Edmonton Board of Health, which in turn forwards them to the Environmental Services for their approval before the water-main can be placed in service.

There has been some confusion regarding the procedures required for water-main acceptance testing in the City of Edmonton. Improper testing can lead to misleading conclusions regarding the acceptability of a new water-main. Some of the issues include unauthorized operation of valves separating new construction from water-mains which are in service (boundary valves), improper pressure testing and leakage determination procedures, misunderstanding of the requirements for preparing water-mains for disinfection, non-uniform distribution of chlorine in newly disinfected water-mains, poor sampling protocol for chlorine analyses, misunderstanding the limitations of chlorine field test kits, poor sampling protocol for bacteriological analyses, and some inexperience and lack of training on the part of the private sector.

The purpose of the guide-lines is to provide a uniform set of acceptance testing criteria and procedures which are presented in a way which is easily understood and can be readily used in the field.

GUIDELINES FOR PRESSURE AND LEAKAGE TESTING

INTRODUCTION

The purpose of pressure and leakage testing a new water-main is to determine if the installation is capable of withstanding ordinary operating transient pressure conditions without failure or excessive leakage at the joints and service connections. The two tests are related and are usually performed at the same time.

To some extent, pressure and leakage testing requirements are a function of the pipe material being used in a water network. Materials which have been used in Edmonton for water distribution include asbestos-cement (AC) AWWA C400 80[1], polyvinyl chloride (PVC) AWWA C900-81, and ductile iron AWWA C110 and C111. Materials used for water transmission include asbestos-cement AWWA C403-84[2], polyvinyl chloride AWWA C905, welded steel with cement mortar lining and coal tar or polyethylene coating AWWA C200-86, and concrete, steel cylinder, prestressed or pretensioned AWWA C301-84 or C303-87.

METHODS

Pressure Testing

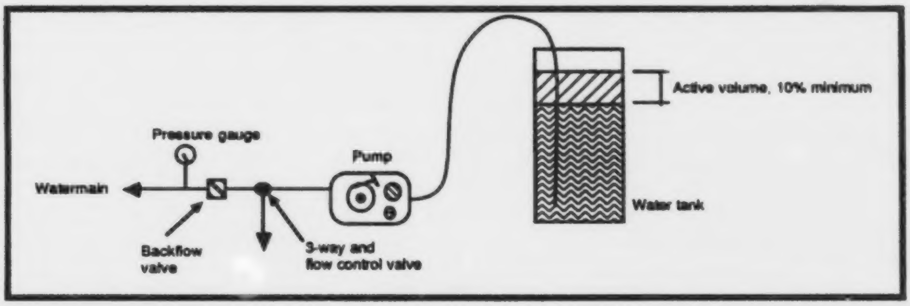
Pressure testing consists of slowly charging a new section of water-main to the distribution system pressure from a boundary main. The valves are then closed on the test section. A tank of water and a purpose-specific pump are used to pressurize the main to the specified test pressure. The main is typically left in this condition for 1 to 3 h. The pressure is normally measured at the lowest elevation on the section of water-main being tested.

Leakage Testing

Leakage testing is normally conducted at the same time as pressure testing using the same apparatus. The test is conducted for 1 to 3 h during which time the pump is periodically operated to maintain the pressure at the specified level. The volume of water added to the water-main from the tank is presumed to be equal to that which has leaked from the water-main during the test. Figure 1 illustrates the conceptual layout of the apparatus required for pressure and leakage testing.

FIGURE 1

Apparatus for pressure and leakage testing



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[1] No longer approved for use in the City of Edmonton

[2] No longer approved for use in the City of Edmonton

Portland cement based materials such as asbestos-cement, concrete, or cement-lined ductile iron pipes are porous and absorb water during the initial charging of the water-main. Consequently, it is usual to require a 24 to 48 hour soaking period to ensure that apparent water leakage is due to joints and service leakage, not the absorption of water by the pipe material.

Required Equipment

The equipment requirements are straightforward, however, it must be dedicated for only one purpose; pressure and leakage testing. The recommended specification is as follows:

a) ANSI Standard B40.1-1974 Grade A, or equivalent, Bourdon tube pressure gauge, range of 0 to 1 400 kPa, with an accuracy of 1% of the full-scale reading, 7 kPa divisions, a minimum diameter of 90 mm, and a scale which can be read in an arc of 270° (Walski, 1984). The pressure gauge calibration shall be certified by the consulting engineer.

b) Hand or motor-powered pump capable of meeting required test pressures including necessary plumbing accessories for preventing backflow and for enabling the flow rate to be controlled (City of Edmonton, 1988). The pump should be provided with a pressure relief valve or should have an upper pressure limit of 1 400 kPa. The pump will be dedicated to water-main pressure testing and will not be used for other construction activities.

c) Water storage tanks will be of non-corrosive material dedicated to water-main testing. The volume of the tank shall not be more than 10 times the allowable leakage for the duration of the test. The depth gauge will be attached to the inside of the tank and will be calibrated for the volume of the tank.

PROCEDURE FOR HYDROSTATIC PRESSURE AND LEAKAGE TESTING

The procedures for pressure and leakage testing are outlined in the following sections and are organized in two parts. General procedures which are independent of pipe material and pipe size are presented first followed by procedures which are influenced by the pipe material or by the size of the pipe.

General Procedures

Procedures which are specific for a given pipe material and/or size are provided after the following section. The following section outlines the procedures which are common to all pipe materials and pipe sizes.

1. Install all water services, air relief services, and blowoffs.
2. Partially or completely backfill the excavation before testing.
3. Wait for concrete thrust blocks to cure: minimum of 3 days in the case of high early strength concrete or a minimum of 7 days in the case of normal concrete.
4. Ensure that corporation stops are open and curb stops are closed.
5. Inform inspectors of the date, time and location of the hydrostatic tests at least 24 h prior to the test time.
6. Open all main valves in the test section.
7. Close all hydrant shutoff valves in the test section.
8. Inform other construction crews or contractors and check that no valves are being operated during the test.
9. Test duration will be 2 h.
10. Maximum length of test sections will be 450 m unless otherwise directed by the City.

11. Ensure air is expelled from the section of water-main by exhausting trapped air through hydrants[3] and special service taps at high points and dead-ends. Air content can be minimized by the following procedures:

    a) lay the pipeline to grade when possible
    b) air relief valves should be provided at all high points.
    c) bleed air from the pipe slowly
    d) fill the water-main at a velocity of less than 0.6 m/s
    e) the diameter of the air relief valve should be such that d/D is 0.01 to 0.1 where d is the diameter of the air relief valve and D is the diameter of the pipe.

12. Raise the water-main pressure to the appropriate test pressure (see section on pipe materials) using either a hand or motor-powered pump located at a hydrant or approved blow-off. The hydrant valve will be completely opened and the flow rate will be controlled by the valve at the pump.

13. Mark the gauge and the level of water in the storage barrel at the beginning of the test. Take care in these marks since it is the basis for calculating water loss.

14. Maintain the test pressure within 20 kPa of the specified test pressure for the duration of the test.

15. Check the pressure after the first 30 min. If the pressure has dropped by more than 20 kPa, stop test and remove air which may be trapped. Repeat the test.

16. During the test, walk along the test section and check for signs of leakage or distress at all exposed appurtenances or fittings.

17. No allowance can be made for services or in-line valves[4].

18. When testing against closed metal-seated valves, add an additional leakage allowance of 0.0012 L/(h · mm of valve size).

Asbestos-cement

AWWA standard C603-78 specifies pressure and leakage testing for AC pressure pipe including distribution and transmission pipelines.

Specific procedures for AC pipe include all of the General Procedures plus the following:

1. Fill the water-main with water 24 h prior to hydrostatic testing

2. Pressurize distribution water-main up to twice the operating pressure at the lowest elevation or to the pressure class designation of the pipe plus 350 kPa, whichever is less.

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[3] Hydrants are not useful for removing all of the air. Air relief valves are essential at high points.

[4] Water-main renewal projects may be exempt under some circumstances.

3. Pressurize transmission water-main to the design pressure (static plus surge) plus 50% more.

4. The allowable leakage can be calculated from the following equation based on two joints per coupling (AWWA C603-78):

$$\text{Allowable leakage} = \frac{ND\sqrt{P}}{896}, \quad \frac{L}{h^{*}100 \text{ couplings}}$$

where N is the total number of couplings divided by 100, D is the diameter of the pipe in mm, and P is the average test pressure in kPa.

Ductile Iron

AWWA standard C600-87 contains the AWWA specifications for pressure and leakage testing ductile iron water-mains and appurtenances including distribution and transmission pipelines.

Specific procedures for ductile iron pipe and appurtenances include all of the General Procedures plus the following:

1. Fill cement mortar lines pipes with water 24 h prior to hydrostatic testing.

2. Test the water-main at a pressure of greater than 125% of the working pressure at the highest point in the test section. The maximum test pressure will be 150% of the working pressure measured at the lowest elevation.

3. Do not operate valves in either direction when the differential pressure exceeds the rated pressure of the valve. Consult AWWA C500 for rated pressures of valves which conform to AWWA standards.

4. The test pressure cannot exceed the rated valve pressure (see AWWA C500) when resilient-seated gate valves or butterfly valves are at the pressure boundary.

5. The allowable leakage can be calculated from the following equation (AWWA C600-87):

$$\text{Allowable leakage} = \frac{LD\sqrt{6}}{2.16 \times 10^{8}}, \quad \frac{L}{h^{*}100 \text{ m}}$$

where L is the length of the test section in m, D is the diameter of the pipe in mm, and P is the average test pressure in kPa.

Polyvinyl Chloride

AWWA C600, C900 and C905 standards do not address in situ pressure and leakage testing of PVC pipe. AWWA C900 and C905 refer the reader to an AWWA manual, M23 (AWWA, 1980b), for information on pressure and leakage testing.

Specific procedures for PVC pipe include all of the General Procedures plus the following:

1. Test pressure will be 150% of the working pressure of 1 036 kPa, whichever is greater, at the point of test but not less than 125% of normal working pressure at the highest elevation on the test section (AWWA,1980b).

2. Allowable leakage will be calculated from the following formula (Uni-Bell, 1982):

$$\text{Allowable leakage} = \frac{ND\sqrt{P}}{1283}, \quad \frac{L}{h \cdot 100 \text{ joints}}$$

where N is the total number of joints divided by 100, D is the diameter of the pipe in mm, and P is the average test pressure in kPa.

Steel

AWWA C206-82 for field welding of steel water pipe addresses hydrostatic pressure testing of new steel water-mains for both distribution and transmission applications. However, no specific details are given except that leaks are not permitted. Cement mortar lines steel pipes need to be filled with water 24 h prior to hydrostatic testing.

Butt welded joints can be tested with a vacuum "look box" which involves placing a soap solution on the joint and then placing a vacuum box over the joint. Bubbles indicate a leak.

Double-welded lap joints can be tested by shop-drilling and tapping a 1/8" or 1/4" NPT hole in the lap or bell end of the pipe and applying 280 kPa of air or other suitable gas to the joint between the two fillet welds. Apply a soap solution to the joint and look for bubbles caused by escaping gas.

Concrete

AWWA C300-82, C301-84, C302-87, and C303-87 are the applicable AWWA standards relating to concrete pressure pipe. These standards refer to an AWWA manual, M9 (AWWA, 1979) for additional information regarding the design and construction of the concrete pressure pipe. The information on pressure and leakage testing is not as specific as the information on ductile iron and AC pressure pipe.

Specific procedures for concrete pressure pipe include all of the General Procedures plus the following:

1. Fill the water-main with water 24 h prior to hydrostatic testing.

2. Test pressure will be 120% of the working pressure.

3. The allowable leakage can be calculated from the following equation:

$$\text{Allowable leakage} = \frac{LD\sqrt{P}}{2.16 \times 10^{6}}, \quad \frac{L}{h \cdot 100 \text{ m}}$$

where L is the length of the test section in m, D is the diameter of the pipe in mm, and P is the average test pressure in kPa.

GUIDE-LINES FOR DISINFECTION

PURPOSE

The purpose of disinfection is to destroy pathogenic micro-organisms which may occupy the water-main after construction is complete. Chlorine is usually used as the disinfectant in either a compressed gas, calcium hypochlorite tablets or made-up solution, or as a sodium hypochlorite solution.

BACKGROUND THEORY

Indicator Bacteria

Pathogens can include a wide variety of organisms including bacteria, viruses, protozoa, and helminths. It is impossible to examine water for every possible pathogen, consequently, a system of indicator micro-organisms has been established which is designed to indicate the presence of fecal pollution in the water. The predominant group of indicator organisms is the coliform group of bacteria. Other indicators have been suggested (Tobin, 1987), however, the speed and simplicity of analysis for the coliform group of organisms have helped to maintain their role in determining water disinfection efficiency. The maximum concentration of total coliforms in new water-mains is 0 colony forming units (CFU) per 100 mL (AWWA C651-86).

Fecal coliform bacteria are a sub-group of the coliform group of bacteria. Therefore, the use of total coliform bacteria for evaluating disinfection efficiency provides a conservative approach to ensuring that water is free of pathogenic micro-organisms. However, it must not be forgotten that certain pathogens such as Giardia lamblia survive much longer in the environment than indicator bacteria and can also be resistant to normal chlorine disinfection practices. Therefore, the results of bacterial indicator testing must always be reviewed in conjunction with other water quality parameters.

Potable water such as is found in water-mains is never sterile. There are many millions of bacteria and other microbes which survive normal water disinfection and can live and multiply in the low nutrient conditions found in water-mains. These organisms do not cause disease but can be a nuisance in a distribution system causing taste and odour complaints and providing microenvironments where disease-causing organisms may survive, protected from the chlorine residual.

A general indicator of the overall biological quality of water is the heterotrophic plate count (formerly total or standard plate count). Heterotrophs are aerobic bacteria which utilize organic carbon for growth. The presence of elevated plate counts in a water system are not a direct cause for concern since they do not usually cause disease. However, elevated plate counts indicate that conditions in a distribution system are suitable for biological activity which is not good for the reasons noted above. Typically, the maximum concentration of heterotrophs in treated potable water has been 500 CFU per mL (Health and Welfare Canada, 1987). Others have suggested that the limit be reduced to 300 CFU per mL (Personal Communication, Edmonton Board of Health, November, 1988).

Elevated heterotrophic plate counts have been associated with carelessness in preventing dirt from entering the pipe during construction, inadequate cleaning of water-mains after construction, and growth on materials used in construction such as jointing gaskets and gasket lubricants (Burman, 1987). Heterotrophic plate count analyses are not performed routinely but are generally used to help identify the source of problems in water-mains when they occur.

Disinfection Kinetics

The rate of disinfection has been traditionally described by the Chick-Watson model [Chick, 1908; Watson, 1908]:

$$\log \frac{N}{N_0} = -kC^n t$$

where C is the chlorine residual, t the time of contact with chlorine residual, N the concentration of bacteria at time t and residual C, N_0 the initial concentration of bacteria, n and empirical factor.

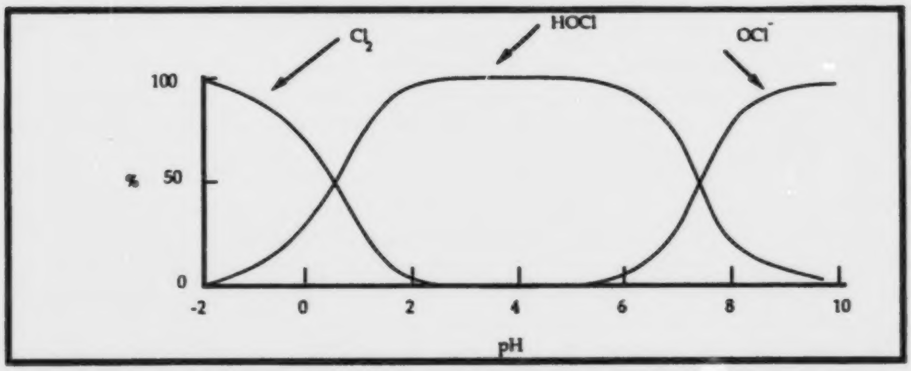
The significance of the Chick-Watson model is that for a given reduction of bacteria, say 99.9%, the Ct product will be a constant (for a specified temperature). The practical implication of this is that, theoretically, the time and concentration can be adjusted to accommodate the necessary Ct to achieve the desired kill.

Free chlorine (hypochlorous acid) acts approximately 100 times faster than monochloramine (combined chlorine). Therefore, the form of chlorine has a major impact on the Ct concept for disinfection of new water-mains.

Basic Chlorine Chemistry

Aqueous chlorine chemistry is strongly pH dependent for both the free and combined forms. The pH dependence of free chlorine is illustrated in Figure 2.

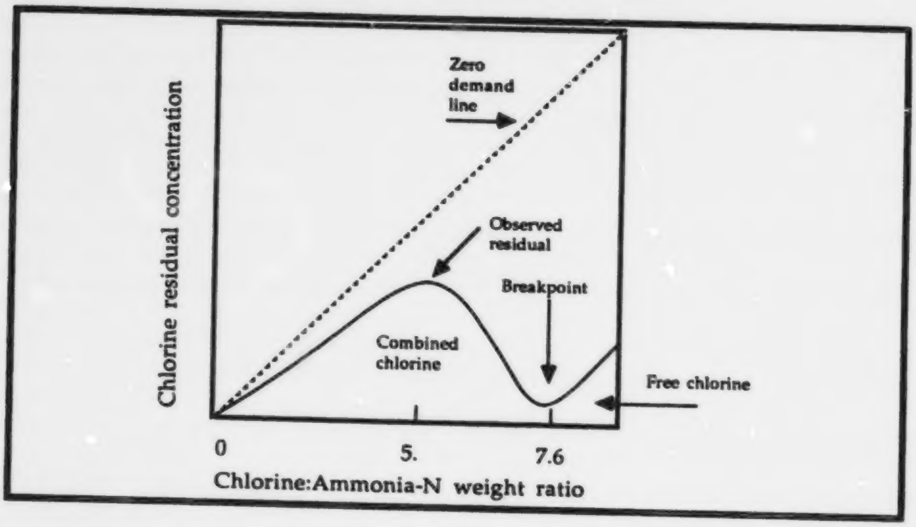
FIGURE 2
Free chlorine species in solution as a function of pH
(After James M. Montgomery Engineers, 1985)



Addition of chlorine in the presence of ammonia creates compounds known as chloramines, or combined chlorine. Chloramines are considered as biocidal as chlorine except that their reaction rates are generally much slower than free chlorine. Again, pH strongly affects the species of combined chlorine which can be found in solution. A process known as breakpoint chlorination, involves the addition of chlorine until all of the ammonia has reacted. A plot of observed total chlorine residual as a function of chlorine added results in a curve with a 'Hump' followed by a 'valley' or the breakpoint. Figure 3 illustrates the breakpoint concept.

FIGURE 3

The breakpoint reaction between ammonia and chlorine (After James M. Montgomery Engineers, 1985)



Chlorine is an oxidant and will react with many organic and inorganic compounds in water or wastewater, sometimes creating nuisance conditions such as taste and odour episodes or post-precipitation of metals which were soluble in their reduced form. These side reactions create chlorine demand and are partly responsible for the decline of chlorine residuals in a water distribution system.

The available chlorine, as Cl_2, depends on the form of the chlorine feed. Chlorine gas is, by definition, 100% available on a mass basis. However, other chlorine-containing chemicals contain only a certain proportion of the chemical mass as available chlorine. Sodium hypochlorite solutions typically contain 6 to 15% available chlorine whereas dry calcium hypochlorite contains approximately 65 to 74% available chlorine (hence the nickname of 'high test hypochlorite', HTH).

The practical significance of these differences is that more sodium hypochlorite is required on a mass basis than calcium hypochlorite for achieving the same chlorine concentration in a water-main.

Analytical Methods

There are a number of methods for the determination of chlorine residuals. The best laboratory method is by means of amperometric titration (Gordon et al., 1988). The best field method for residual chlorine from 0 to 10 mg/L is the colorimetric N,N-diethyl-p-phenylene-diamine (DPD) procedure (Gordon et al., 1988). The phenylarsine oxide (PAO) drop titration method is used for high concentrations (10 to 200 mg/L) of chlorine. It is not a precise analytical method but is satisfactory for the purpose of primary disinfection of water-mains provided more accuracy is not expected than is implied by the method.

DISINFECTION SPECIFICATION

Performance criteria for water-main disinfection has been specified using the American Water Works Association (AWWA) standard C651-86 (formerly C601-68). This standard comprehensively describes the minimum procedures to be followed when preparing a water-main for disinfection, disinfecting the water-main, for testing chlorine residual, and for bacteriological sampling. It refers to the latest edition of Standard Methods (APHA, AWWA, WPCF, 1985) for detailed methods for testing water quality and chlorine residual.

PROCEDURES

Disinfection consists of the following four tasks:

1. Preventing contamination of the new pipe during shipping, storage, and construction.
2. Flushing the water-main to remove loose debris and dirt which may have entered the water-main during construction. **Do not flush** if hypochlorite tablets have been placed.
3. Chlorination of the water-main to destroy pathogenic micro-organisms.
4. Bacteriological testing of the disinfected water to ensure that the microbiological water quality is adequate.

Water-main Preparation

Water-main flushing is the most common form of preparation for disinfection except where hypochlorite tablets have been used. The flushing flow rate should be sufficient to achieve a minimum velocity of 0.8 m/s. Table 1 summarizes the flow conditions required to achieve this velocity for various pipe sizes.

TABLE 1

Required flow and number of openings to flush water-mains**

(After AWWA C651-86)

Pipe diameter, mm	Required flow for 0.8 m/s velocity, L/s	Size of tap, mm			Number of 2.5" hydrant nozzles*
		25	40	50	
		Number of taps			
100	6.5	1	-	-	1
150	13	-	1	-	1
200	26	-	2	1	1
250	38	-	3	2	1
300	57	-	-	2	2
450	100	-	-	4	2

* With 280 kPa residual pressure, a hydrant flowing to atmosphere will discharge 63 L/s from a 2.5" nozzle and 158 L/s from and 4.5" steamer nozzle.

** Assumes 280 kPa residual main pressure.

Larger water mains can be broom swept as an alternative to flushing. Care must be taken in removing the sweepings from the pipe before filling.

When care has been exercised in the shipping, storage, and construction of new water-main, the above procedures will normally remove contaminates which remain in the pipe. However, on occasion, flushing will not be satisfactory and a more thorough cleaning method will be required. This will typically involve foam swabbing of the water-main. A foam swab is a bullet-shaped piece of polyurethane foam, available in a variety of densities, which is cut approximately 50 mm larger in diameter than the pipe to be cleaned. The swab is usually launched through hydrant leads and pushed through the new water-main hydraulically. The soft foam gently scours the walls of the pipe, removing stubborn contaminants. It should be certain that the swab is not compressing contaminants into the pipe joints. The equipment and procedure for foam swabbing is somewhat specialized and should only be undertaken by properly trained personnel AWWA, 1986). Some authorities require foam swabbing prior to acceptance of new water-mains (AWWA, 1986; Burman, 1987).

Chlorination

The choice of chlorine chemical is limited to sodium hypochlorite or calcium hypochlorite. There are 3 accepted methods for adding chlorine chemicals to new water-mains (AWWA C651-86):

1. Calcium hypochlorite tablets placed during construction. This method provides an average dose of 25 mg/L.

2. Continuously fed sodium or calcium hypochlorite solution. This method provides a minimum chlorine residual of 10 mg/L for 24 h.

3. A slug of a high concentration chlorine solution. This method provides a high concentration of chlorine, 100 mg/L for a contact time of greater than 3 h.

Calcium Hypochlorite Tablets

Calcium hypochlorite tablets are placed in the water-main during construction. The tablets are usually 5 g (3.25 g as available chlorine) and are placed at the beginning of the new main, at 150 m intervals, each hydrant lead, each hydrant, and other appurtenance. The number of 5 g tablets can be calculated from:

$$\text{No. of tablets} = 6.28 \times 10^{-6} d^2 L$$

where d is the pipe diameter in mm and L is the length of pipe being disinfected in m. This method cannot be used with solvent-welded plastic or screwed-joint steel since the chemicals are incompatible with calcium hypochlorite.

The water-mains slowly filled after construction keeping the velocity below 0.3 m/s to prevent disturbing the tablets. Removal of air from the water-main requires particular care as was noted in the section on pressure and leakage testing above. The contact time should be 24 h at water temperatures greater than 5°C, and should be increased to 48 h when the water temperature is less than 5°C.

Continuous-feed

A solution of calcium hypochlorite or liquid bleach (sodium hypochlorite) may be continuously fed as the water-mains discharged at the end of the section being disinfected. The objective is to fill the water-main with a uniform concentration of chlorine so that at the end of 24 h, a minimum chlorine residual of 10 mg/L remains.

The following equipment is required when performing continuous-feed hypochlorination:

1. A gasoline or electrical chemical-feed pump designed for feeding chlorine solutions. The usual type is a positive-displacement diaphragm pump which is available in two styles; mechanically or hydraulically actuated. The chemical feed rate can be adjusted by altering either the stroke length or stroke speed.

   Maintenance of chlorine feed equipment is important since the high pH which characterizes hypochlorite solutions causes valve seats to fail from scaling, hardening, or swelling. Calcium carbonate scale can be removed from the equipment by passing 5% hydrochloric [muriatic] acid through equipment. **Be sure to flush chlorine from equipment with water first since chlorine gas can be produced.**

2. All hose and fittings should be constructed of material which is oxidant resistant and strong enough to withstand the maximum pressure of the chemical-feed pump.

3. A high concentration chlorine test kit should be used to make chlorine residual determinations at the discharge end of the water-main being tested.

The chlorination procedure consists of the following steps:

1. During construction, calcium hypochlorite tablets may be placed in the water-main. This permits a strong chlorine solution to flow down the pipe during filling, penetrating annular spaces at pipe joints. See the above section for the procedure for placing calcium hypochlorite tablets.

2. Prepare a 1% chlorine feed solution from dry calcium hypochlorite or sodium hypochlorite bleach solution. Refer to Table 2 for information on the amounts of solution which are required for various pipe sizes. Wear protective clothing, face shield, and filter face mask. **CHLORINE IS EXTREMELY HAZARDOUS.** Do not store chlorine near acid or petroleum products. **EXPLOSION HAZARD.**

TABLE 2

Chlorine required to produce 25 mg/L concentration in 100 metres of water-main
[After AWWA C651-86]

Pipe diameter, mm	100% chlorine^ g/100 m	1% chlorine solution *^ L/100 m
100	20	2.0
150	45	4.5
200	81	8.1
250	130	13
300	180	18
450	323	33

* 1.5 kg $Ca[OCl]_2$ per 100 L

^ adjust for available chlorine of chemical used

3. Ensure all boundary valves are closed. Use a water distribution system drawing to highlight all valves and pipes involved in the area to be disinfected.

4. Set-up chlorination equipment in such a way that the feed point is not more than 3 m downstream from the beginning of the new water-main.

5. **DO NOT** use fire hydrants for chemical feed. The high concentration chlorine will damage the hydrant.

6. Notify Water Network Maintenance Dispatch and open one boundary valve and the discharge valve permitting water from the distribution system or other approved source to flow through the new water-main at a constant, measured rate.

7. Adjust the chemical feed pump rate to produce a chlorine dose of 25 mg/L free chlorine. For example, if the water flow rate in the water-main is 40 L/min, the chemical feed pump should be set to deliver 1.0 g/min (100 mL/min if a 1% solution is being used.)

8. Frequently monitor the discharge location for chlorine residual using an approved field test kit.

9. Once the 25 mg/L residual has been achieved, stop the discharge and chlorine feed. Retain the chlorinated water in the test section for 24 h or more.

10. Operate all valves and hydrants in the test section to ensure are disinfected.

11. After 24 h check the free chlorine residual. If the residual is less than 10 mg/L, flush and rechlorinate.

SLUG

The slug method involves slowly flowing a slug of high concentration chlorine, 100 mg/L, through the water-main, exposing all surfaces to the chlorine for a minimum of time of 3 h.

The procedure for slug chlorination is as follows:

1. During construction, place calcium hypochlorite tablets in the water-main. This permits a strong chlorine solution to flow down the pipe during filling, penetrating annular spaces at pipe joints. See the section on calcium hypochlorite tablets for details.

2. Ensure all boundary valves are closed. Use a water distribution system drawing to highlight all valves and pipes involved in the area to be disinfected.

3. Set-up chlorination equipment in such a way that the feed point is not more than 3 m downstream from the beginning of the new water-main.

4. **DO NOT** use fire hydrants for chemical feed. The high concentration chlorine will damage the hydrant.

5. Notify the inspectors and open one boundary valve and the discharge valve permitting water from the distribution system or other approved source to flow through the new water-main at a constant, measured rate.

6. Adjust the chemical feed pump rate to produce a chlorine dose of 100 mg/L free chlorine. For example, if the water flow rate in the water-main is 10 L/min, the chemical feed pump should be set to deliver 1.0 g/min (100 mL/min if a 1% solution is being used).

7. The flow of the slug which moves along the water-main should be controlled so that the pipe and appurtenances are exposed to 100 mg/L for 3 h.

8. Frequently monitor the free chlorine residual in the slug using an approved field test kit. If is falls below 50 mg/L, move the chlorination equipment to the head of the slug and start the flow again, increasing the concentration to 100 mg/L.

9. Operate all valves and hydrants as the slug passes to ensure they are disinfected.

Dechlorination

Once chlorination has been satisfactorily completed, water-mains need to be flushed to remove the high concentration of chlorine which remains in the water. This is necessary to prevent damage to the pipe material and fittings and to ensure heavily chlorinated water does not enter homes. However, in some special circumstances, dechlorination may be required before discharging the chlorinated water to the environment to meet the regulatory requirements of the City of Edmonton or Alberta Environment. This requirement may arise because chlorine is very toxic to organisms in an aquatic environment. Consequently, the disposal of heavily chlorinated water needs to be evaluated at each location. Flushing normally directs water into the storm sewer system which in turn directs the water to either a storm water management facility or to a trunk storm sewer which eventually discharges into a water course (or in the case of combined sewers, to the wastewater treatment plant).

An alternative method, though less desirable, is disposal by flushing to the sanitary sewer system. However, permission needs to be obtained since the large volumes of highly chlorinated water may cause hydraulic problems in the sewer system and the chlorine may be toxic to the biological processes at the treatment plant.

Dechlorination, if required is best performed by adding a neutralizing chemical to the chlorinated water as it is flushed from the system and before it enters the receiving environment. Table 3 provides information on various chlorine neutralizing chemicals which can be used.

TABLE 3

Chemicals for neutralizing heavily chlorinated water
(After AWWA C651-86)

Residual chlorine concentration mg/L	Sulphur dioxide (SO_2) mg/L	Sodium bisulphate ($NaHSO_3$) mg/L	Sodium sulphite (Na_2SO_3) mg/L	Sodium thiosulphate ($Na_2S_2O_3 \cdot 5H_2O$) mg/L
1	1.0	1.5	1.7	1.5
2	2.0	3.0	3.4	3.0
10	10	15	18	6.0
50	50	75	90	75

NOTE: 1 mg/L = 1 g/1000 L - 1 g/m^3

Sampling

The purpose of sampling a water-main is to obtain information on the water quality in the pipe. The underlying assumption which is important is that the sample is **representative** of the water in the water-main. Good sampling procedures are intended to provide the best opportunity for obtaining a representative sample.

Chlorine

Sampling for chlorine residual includes two basic tests: high concentration chlorine residual at the beginning and end of the primary disinfection period; and sampling for low concentration chlorine residual once the water-main has been flushed. The best location for sampling residual chlorine during the primary disinfection period is at the discharge blowoff and at the copper services intended for bacteriological sampling. Low concentration chlorine can be determined at the bacteriological sampling locations, at an approved blowoff, and in the last resort, at fire hydrants.

Bacteriological

Sampling for bacteriological water quality parameters requires a great deal of care since it is very easy to contaminate a sample. Of utmost importance is to recognize that bacteria are organisms which live everywhere including air, soil, water, clothing, and all parts of the human body. Therefore, good sampling protocol is necessary to ensure reliable results. The following procedure will ensure reliable results are obtained in the laboratory [APHA, AWWA, WPCF, 1985; AWWA, 1986]:

1. Obtain approved bacteriological sample bottles.
2. Keep the bottles closed until immediately prior to obtaining the sample. These bottles are sterilized and contain a tablet of dechlorination reagent. **NEVER** rinse the sample bottles.
3. **DO NOT** sample from fire hydrants. Use a copper service or blowoff especially designed for sampling the new water-main.
4. Flow the sample line at high rate for a minimum of 5 min to flush the line thoroughly before sampling.
5. Reduce the flow rate in the sample line, but maintain a water flow.
6. Carefully break the seal on the sample bottle cap and unscrew. Take great care not to contaminate the cup or the neck of the bottle with fingers or dirt.
7. Avoiding splashes, cut the water stream with the sample bottle and fill it until there is a head-space of approximately 20 mm in the sample bottle roughly to the shoulder of the sample bottle]. Replace the cap securely.
8. Shake the bottle 5 times to help the dechlorination reagent dissolve.
9. Label the bottle clearly and complete all necessary information including time, date, location, water type, and required analyses.

10. Bacteriological samples cannot be stored. If the sample cannot be delivered to the laboratory within 1 h of sampling, use an iced cooler for storage during transport. In no case, should the samples be delivered to the laboratory more than 6 h after the sample was collected. **NEVER** leave sample bottles in the sunshine or expose them to elevated temperatures.

TECHNOLOGY TRANSFER

The guide-lines described in this paper have been introduced this year for the first time in the City of Edmonton. To facilitate the use of these guide-lines and to ensure uniform application, the City of Edmonton sponsored two transfer-training seminars: one for consulting engineers and the other for construction forces. At this time, there has been positive feed-back on the benefits of this type of seminar for introducing uniform water-main acceptance testing standards. It is anticipated that this will continue in the future to ensure that a high standard of new water-main construction is maintained in the City of Edmonton.

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THE USES OF LINK PIPE AND SNAP-LOCK FOR REHABILITATION OF UNDERGROUND SEWER SYSTEMS

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INTRODUCTION

Originally conceived in 1980 by Lem Meimets, a Toronto engineer, to fill a need left by polyethylene relining and grouting. Excavation was the only alternate with its disadvantages, not the least of which, in Ontario, is traffic tie-ups.

Existing systems could not repair voids, broken pipe or longitudinal cracking, and lining, at that time, left an annular space. The requirements were:

Simple equipment
Utilization of Arch Support Theory
Placement method
Wall thickness of liner material
Type of material
How to hold in place

Disadvantages at that time were that the majority of existing inspection equipment was very large and heavy; video cables were large in diameter and consequently very heavy. Tow cables had to be very strong and were therefore steel, and tow winches were encouraged to be power driven.

Link-Pipe was originally designed with large links to be pushed horizontally. The jacking system to expand it was very heavy (over 100 lbs.) and it was difficult to lock initially due to lack of methods to obtain exact Inside Diameter. Additionally, with Canada in recession, money was tight and rehabilitation was "unfashionable". However, gradually major centres saw the possibilities and jobs were carried out. These included the City of Toronto; City of Hamilton; New York State; City of Mississauga; City of Portland, Oregon and Washington, D.C. Only round pipe was attempted and the remote jacking method was exclusively used. A standard wall SDR PVC pipe was used with a 304 stainless steel version being used for particularly hostile environments.

As previously stated, the jacking assembly was expensive, very ungainly and the hydraulic hoses heavy. A history was slowly being compiled and with the installation (manually in large diameter pipe), it was found that the hinges actually could resist the links from sealing properly. The hinges were designed to corrode away within 12 months anyway and were strictly for installation. It was decided to install without hinges. Production rates rose by 300 percent. Consequently now, jobs where man access is available are installed very quickly and simply.

A Canadian manufacturer of PVC pipe forms PVC to any required shape in its ovens, and many jobs have been carried out in various sectional sizes and lengths.

Link-Pipe was originally only available for 15-inch diameter and up. Due to this disadvantage and knowing full well that over 70 percent of North American sewer system mileage is 8-inch diameter, a spiral sleeve was conceived and a patent applied for to fill the 8-inch to 15-inch range.

Extended to be from 8 inches to 30 inches with placement using an expandable rubber plug, Snap-Lock now existed to service a full size range, with 4-inch size under development.

A major disadvantage of Snap-Lock is that it looks like a tin can. Actually, made of stainless steel with much thicker walls, it is anything but a tin can. The design is such that for 8-inch diameter with minimum wall thickness it will withstand an hydraulic buckling force of 140 feet of water. Additionally, material wall thicknesses can be increased as required by individual sleeves; i.e., if line cover increased from 12 feet to 40 feet, the sleeve can be increased from 24 to 22 to 20 gauge if so required.

The projected life of Link-Pipe is equal to that of SDR PVC pipe; i.e. in excess of one century. When stainless steel is used, series 304 is utilized as specified for use in all sewage treatment plants and wet wells. 304 is magnetic but there are instances where a non-magnetic material is used.

Arch Support is the key to Link-Pipe/Snap-Lock as it is to any successful pipe design. The requirement of springline support gives additional strength and when added to the hoopstress generated by the gasket compression, considerable strength is available. If failure occurs, it can be in the form of buckling. The buckling failure loading can be calculated. With the design loading being 5-10 times the depth of cover on the pipeline, failure is unlikely. Support of a structurally cracked pipe basically requires enough strength to stabilize the structure. The original pipe will then form part of an integrated structure with the Snap-Lock/Link-Pipe.

Using the Ethafoam gasket seal to ensure no annular space, infiltration can be sealed to considerable pressures. Most engineers prefer this to methods leaving an annular space, due to the structural considerations; and the simplicity of the method is appealing.

Typical locations where Snap-Lock has been used are longitudinal cracks, circumferential cracks, multiply cracking, infiltration, and voids. As can be seen, normally some of these locations would previously have required excavation. Those with running infiltration through longitudinal cracks are almost impossible to grout and would therefore require more expensive renovation methods or excavation.

DIFFICULTIES

There are some difficulties at times with any renovation method and Link-Pipe is effected by some:

Misaligned Joints - making it difficult to gain access on occasion.

Sags - placement is best accomplished with a pan-and-tilt camera where you can watch the crown. Also, Snap-Lock cannot remove the sag effect from hydraulic evaluation; i.e. a constriction due to material buildup.

Protruding Services - a problem for most maintenance methods and must be cut off in small diameter lines, particularly if a sleeve is to be placed over the service connection. Where there is damage around a connection, a repair can normally be carried out; however, if a pan-and-tilt camera inspection shows damage in the lateral, it can encourage excavation.

Deflection - a problem in two ways:

(1) it can limit access due to required clearances.
(2) if apparent at the location of a proposed repair, it becomes an individual decision with regard to the finished product.

At this time, repairs are limited to within 10 percent deflection. Individual queries are, however, evaluated. Repairs have been made with greater distortion but it is felt to be better to increase the number of installations that are satisfactory to all concerned. To this end, there are a range of in-house manufactured deflection meters attached to the camera that can measure, at will, to 1 mm accuracy on diameter.

Voids - potentially a problem and expansion loads are very carefully regulated. There are situations in cracked pipe where the broken pipe can be expanded. In reality this doesn't often occur due to the loading of the backfill. However, it is critical to strengthen known voids to remove the risk of collapse or impact loading on liner pipe. Link-Pipe has been used in this way on several U.S. contracts.

Changes is Diameter - It was mentioned earlier that misaligned joints can be a problem. However, some can be sealed by correct use of the gasket and compressions generated. One particular job has been carried out where access was through a 90-degree catch pit past a 45-degree bend and some 45 feet to a 5-degree misaligned joint with 1-inch difference in diameter. This joint was sealed against running infiltration. Many misaligned joints have slipped to the point that sleeve insertion is not possible.

COSTS

Link-Pipe is costed on a per sleeve basis in its use with the installation vehicle being costed on an hourly basis. The pipeline in question is required to be clean to obtain a good seal on the sleeve; however, installation can be carried out with normal sewage flows. This doesn't mean 80 percent of diameter flow depths.

This method of costing causes the first sleeve to be the most expensive, requiring all preparatory work and initial video plus mobilization. The second sleeve onwards is relatively inexpensive. Single 4-foot sleeves have been placed for as little as $850.00 and as much as $2,500.00. In normal conditions, it is no longer necessarily cheapest to excavate. Client cities seem to be finding everything below 8 feet in depth under pavement or in isolated locations are cheaper to evaluate for insitu renovation. Link-Pipe becomes an alternate for use as a repair method, as in grouting, PE Lining, Insituform and Durapipe.

INSTALLATION

Link-Pipe installation is carried out using a specialized video inspection truck with on-board compressed air. Using special Logi-ball plugs, the sleeve is towed into position located by a footage counter and visually, and then expanded and clipped.

Should placement of the sleeve be over a service connection, a location cartridge is placed initially in the service by manipulator. After sleeve installation, a traction hydraulic cutting device locates the cartridge and automatically aligns on the theoretical centreline of the lateral and retrieves the cartridge. This cutting assembly will also cut off protruding laterals where necessary with no chipping or cracking. This device is special in that it is not towed into place; it is automated and computer controlled. The head has 10-way movement to 1 mm. It utilizes slurry injection techniques and, not being reliant on only very high pressure water or mechanical methods, can cut steel. Its manufacture specification require it to be able to cut a 2-inch diameter billet of steel. While being very new, this unit has the potential to change some maintenance functions in pipe systems.

Also, a pan-and-tilt camera is invaluable as it allows detailed examination of cracks or voids and laterals, remote controlled focus being very useful. It can check pipe walls before and after lining, including all cutouts. Lateral inspections are carried out by miniature cameras that can consistently reach the main from the cleanout in a residence and are very useful, cameras being available to pass 90-degree bends in 3-inch diameter pipe and still travel 110 feet to the main.

Roots are removed using high pressure rotary spinners running at 4,500 psi. Removal is complete and debris floats away with the preparation for lining, carried out quickly and simply.

CONCLUSION

Our company feels that rehabilitation equipment is now sophisticated enough to consistently use renovation methods successfully and less expensively than previously. Now, at long last, rehabilitation is becoming very much cheaper than replacement, with a lifetime of at least the original designed pipe. Municipalities can obtain more repair for their dollar, engineers can be more ingenious and our construction industry is going to change.

WATER TREATMENT AND DISTRIBUTION OPERATION DURING THE 1988 SPRING RUN-OFF IN EDMONTON

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Episodes of objectionable taste and odour in the Edmonton water supply have occurred regularly during the spring thaw. This phenomenon has been studied and reported extensively. (Walker et al, 1986; Hrudey and Associates, 1986).

Subsequent to the intensive studies of 1986 a Process Development Team was set up and the Water Treatment Laboratory launched a Quality Control/Quality Assurance program. The laboratory has also implemented a monitoring program using a UV spectrophotometer and a gas chromatograph to detect and evaluate taste and odour problems (Inni Rashid, 1987). These techniques are now used on a day to day basis by water treatment plant operators for process control. Operating procedures for taste and odour control were developed for both water treatment plant sites. (Table 1, Table 2). Ironically, the following intensive study, improved monitoring and formulation of operating procedures, the spring thaws of 1987 and 1988 were virtual non-events with respect to raw water quality. This was due to low winter snow accumulation and gradual melting.

The source of supply for the City of Edmonton is the North Saskatchewan River, which rises in the Columbia Icefields and is fed by a number of tributaries including the Clearwater, Brazeau and Nordegg Rivers. (See Figure 1).

FIGURE 1

The North Saskatchewan River Basin Upstream of Edmonton

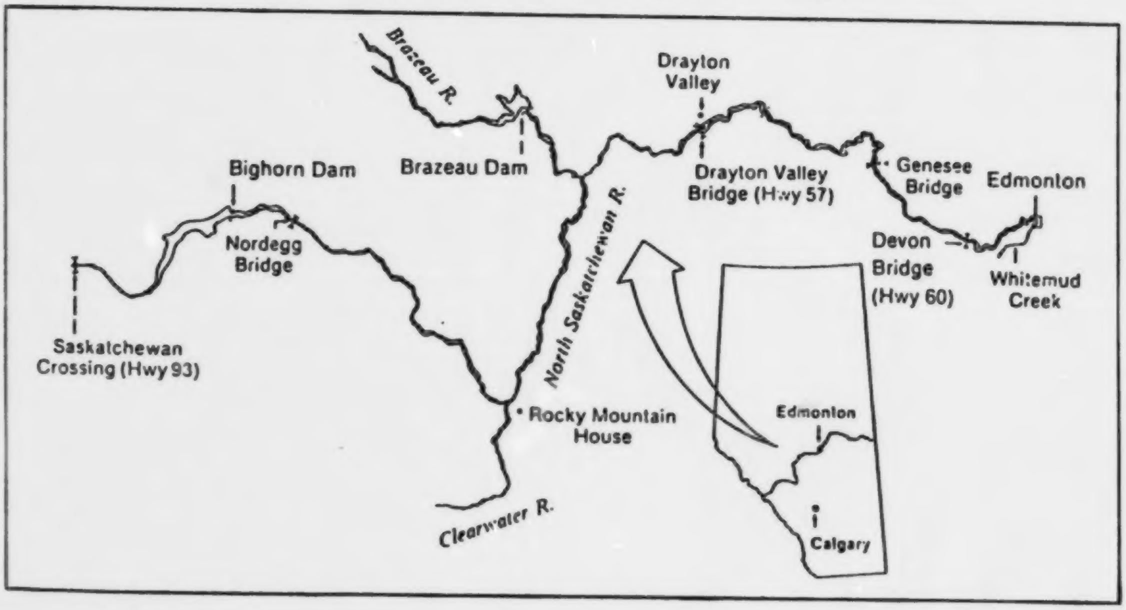

Brazeau R.
Drayton Valley
Bighorn Dam
Brazeau Dam
Drayton Valley Bridge (Hwy 57)
Genesee Bridge
Edmonton
North Saskatchewan R.
Nordegg Bridge
Devon Bridge (Hwy 60)
Whitemud Creek
Saskatchewan Crossing (Hwy 93)
Rocky Mountain House
Edmonton
Calgary
Clearwater R.

TABLE 1

Rosedale Water Treatment Plant
Taste and Odour Control Procedures

DATE: 28/Oct/86

CONDITION				RESPONSE							
	DESCRIPTION	RAW WATER		MONITORING	CHEM FEEDS	OBJECTIVE					OTHER
						Unit	C 1-1 2-1	C 1-2 2-2	Treated	#3 Plant	
1	URBAN RUNOFF	VO	> 10	Raw & Treated GC min. 2/shift	-Carbon 5ppm If Winter Temp. Forecast To Go Above Freezing	VO	< 30	< 30	< 20	< 20	Shut #3 Plant If Treated Obj. Is Exceeded
	- Due to rain	Turb	< 50	Jar Test As Required	-Carbon 5ppm If Rain Forecast	Turb	< 5	< 10	< 1.0	< 1.0	Increase Carbon to 20 ppm If Raw GC >10
	- Due to snow melt	Color	< 10	Every4hr.Raw/C1-1/C1-2/Treated		Color	< 5	< 5	< 5	< 5	Deploy Absorbant Boom To C1-1 & C2-1
		UV	< 10	Treated UV min. 2/Shift	-Maintain Oxidant Residuals As Demand Increases	UV	< 10	< 10	< 5	< 5	Treated water greater than 20 - shut plant down.
2	SPRING RUNOFF URBAN AND RURAL	VO	> 10	Raw & Treated GC min. 2/shift	-Carbon 20ppm/10ppm If GC<10	VO	< 30	< 30	< 20	< 20	-Call for Lab support
		Turb	> 50	Jar Test min. 2/Shift	-Reduce first stage pH 6.5 - 6.8	Turb	< 5	< 10	< 1.0	< 1.0	- Select centre river intake
	- Due to snow melt	Color	> 10	- Use color, UV and Turb as indicators		Color	< 5	< 5	< 5	< 5	Cut pumpage #1 Plant to 40 MLD and #2 Plant To 60 MLD If Treated Objective Will Be Exceeded
		UV	> 10	Color/UV min.Every 4 hr.	-Maintain Oxidant Residuals As Demand Increases	UV	< 10	< 10	< 5	< 5	
3	OIL/CHEM SPILL - Due to overturned truck - Due to spills from Unknown Sources	N/A		Call Lab Staff For Continuous GC Availability -Increase GC monitoring during Event	-Feed carbon at 10 ppm After Plant Start-Up In Case Raw Water Was Contaminated -Reduce when threat is over	Volatile organics levels less than 20 ppb					-Shut All Plants Down To Avoid Contamination -Notify On-Call Staff/Supr. As Soon As Possible -Notify Environment , Drainage & Water Dispatch
4	ODOUR -Odour Detected In Plants From Unknown Source	N/A		Call Lab Staff For Continuous GC Availability -Increase GC monitoring during Event	-Feed carbon at 20 ppm -Apply carbon locally on surface films -Reduce when threat is over	No Objectionable Odours					-Notify On-Call Staff/Supr. As Soon As Possible -Check Plant Processes For Causes -Isolate By Plant If Possible
5	PRECAUTIONARY -Surface Film On Clarifiers	N/A		Call Lab Staff To Check GC	-Feed carbon at 5 ppm Apply Carbon To Surface Films Apply Absorbant Boom Or Pads As Required	PRECAUTIONARY MEASURES SHOULD BE EMPLOYED WHEN RISK OF MINOR CONTAMINATION EXISTS DUE TO WORK CARRIED OUT DURING CONSTRUCTION OR ROUTINE EQUIPMENT MAINTENANCE SUCH AS: PIPE REPLACEMENT,OIL CHANGES,GENERAL PAINTING AND CLEANING					

DOC: ROS:T&02.WK1
REV: 89/02/27

TABLE 2
E.L. Smith Water Treatment Plant
Taste and Odour Control Procedures

DATE: 23/Jun/87

DESCRIPTION OF CONDITION	PROCESS STATUS					MONITORING RESPONSE				CHEMICAL FEED RESPONSE		OTHER RESPONSE
	Unit	Raw	Eff C1/C2	C3 Eff	Potable	** GC	UV/Color	Odour	Jar Tests	Carbon	ClO2	
1 PRECAUTIONARY MEASURE Feb. 15 - Oct. 15	UV	<10	X	X	X	1 Raw/day 1 Trt/day	2 Raw/Sh 2 Trt/Sh	6/Shift	As Required	2 ppm	0.10 - 0.19 and Total ox<0.4	See conditions below for specific response.
	Color	<15										
	Turb	<50	X	X	X							
	VO	<5										
2 TURBIDITY (NTU)	Turb	>50	-	-	-	1 Raw/day 1 Trt/day	3 Raw/Sh 3 Trt/Sh	6/Shift	At least 2 per shift or when Turbidity Change >10	5 ppm	0.10 - 0.19 and Total ox<0.4	Step-up clarifier monitoring
	Turb	>50	-	>15	-							Plant to Waste until Recovery*
	Turb	>50	-	-	>1							Plant to waste*
3 UV ABSORPTION (Humic Acid Units)	UV	>15	<5	<5	<5	Potable > 8 2 Raw/day 4 Trt/day	3 Raw/Sh 2 Int/Sh 3 Trt/Sh	12/Shift	At least 3 per shift with UV as criteria for Jar Test.	5 ppm	0.10 - 0.19 and Total ox<0.5	Monitor closely
	UV	>15	>5	>5	>5					10-15 ppm		Step-up monitoring
	UV	>15	>10	>10	<10					15-20 ppm		Step-up monitoring
	UV	>15	>10	>10	>10					25 ppm		Plant to waste* cut pumpage
4 COLOUR (TCU)	Color	>20	<2	<2	<2	Potable > 5 2 Raw/day 4 Trt/day	3 Raw/Sh 2 Int/Sh 3 Trt/Sh	12/Shift	At least 3 per shift with color as criteria	6 ppm	0.10 - 0.19 and Total ox<0.5	Monitor closely
	Color	>20	>2	>2	>2					10-15 ppm		Step-up monitoring
	Color	>20	>4	>4	<5					15-20 ppm		Step-up monitoring
	Color	>20	>5	>5	>5					25 ppm		Plant to waste* cut pumpage
5 GC (VO) (Volatile organics - ppb)	VO	>2	-	-	-	Potable > 8 4 Raw/day 4 Trt/day	2 Raw/Sh 2 Int/Sh 2 Trt/Sh	12/Shift	As Required	10 ppm	0.10 - 0.19 and Total ox<0.5	Step-up monitoring
		-	-	-	3 - 5					15 ppm		Step-up monitoring
		>20	-	-	-					15-20 ppm		Step-up monitoring
		-	-	-	>10					25 ppm		Plant to waste* cut pumpage
6 GROSS POLLUTION REPORTED (Chemical or Oil Spill)	N/A					4 Raw/day 4 Trt/day	6 Raw/Sh 2 Trt/Sh	12/Shift	As Required	15 ppm	0.10 - 0.19 and Total ox<0.5	Step-up Raw Water monitoring Once spill is detected, follow above conditions to respond.
7 VISUAL OBSERVATION OF OIL IN PLANTS	N/A					4 Trt/day	8 Trt/Sh	12/Shift	As Required	>15 ppm	0.10 - 0.19 and Total ox<0.5	Step-up Treated Water monitoring: When condition identified follow above criteria to respond.
8 INCREASED CHLORINE DIOXIDE DEMAND Not Meeting Residual Due to Total Oxidants	N/A					2 Raw/day 4 Trt/day	3 Raw/Sh 3 Trt/Sh	12/Shift	As Required		Total Oxidant <0.5	Increase monitoring of all parameters. Ask Lab to increase frequency of yield determinations.
9 ODOUR	Odour is detected in treated water					Call Lab to do immediate GC.	6 Raw/Sh 6 Trt/Sh	12/Shift	As Required	25 ppm	0.10 - 0.19 and Total ox<0.5	If field and site reservoirs in full condition, consider wasting.*

The quality of the glacial meltwater is good but the adjoining tributaries, particularly the smaller ones, can carry a substantial organic loading.

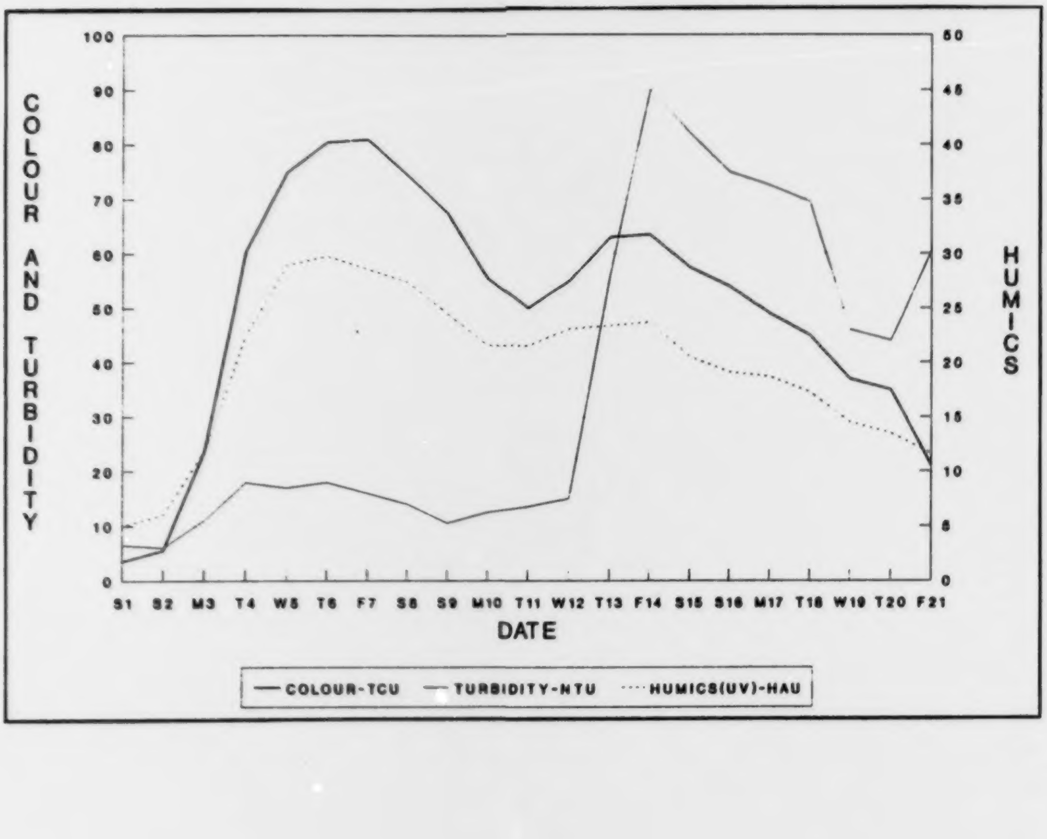
Spring run-off results from the melting of large accumulations of snow and ice in the rural area upstream of Edmonton. The meltwater carries a natural organic loading which arises from decaying vegetable matter and is commonly referred to as humic substances. This material creates musty or earthy odours and taste, and imparts a yellowish brown colour to the water.

The severity of the spring run-off can be represented by the following quality parameters of the raw water:

(a) Colour in True Colour Units (T.C.U.)
(b) Concentration of humic substances in Humic Acid Units (H.A.U.)
(c) Turbidity in Nephelometric Turbidity Units (N.T.U.)

The most difficult water to treat is that which has high colour, high concentrations of humic substances and low concentrations of suspended particles (i.e. low turbidity). Since these conditions arise from snow melt the water temperature is low which hampers chemically aided coagulation and other chemical and physical reactions in the water treatment process. (See Figure 2).

FIGURE 2
Raw Water Quality - April 1989



On Saturday April 1st the raw water was displaying typical winter characteristics as defined by the Process Development Team (See Table 3).

TABLE 3

Raw Water Type	Typical Raw Water Quality Parameters								Water Production Capabilities – ML/d					
	Turb NTU	Colour TCU	U.V. @ 200 nm	T Alk mg/L	T Hard mg/L	TOC mg/L	Temp C	pH	Plant 1	Plant 2	Plant 3	Total Ross	Plant 4	Total System
Type I Winter	3	4	5	130	160	3	0.6	8.1	60	105	50	215	140	355
Type II Spring Upstream Runoff	10	90	16	110	130	N/A	0.7	8.0	45	65	0	110	90	200
Type III Spring Breakup	150	20	5	120	150	N/A	3	8.1	55	80	0	135	110	245
Type IV Summer Rains	80	20	16	130	150	10	17	8.2	80	120	50	250	180	430
Type V Summer Normal	10	10	12	110	150	10	20	8.3	100	150	75	325	190	515
Type VI Summer Flood	350	60	20	120	150	N/A	17	8.2	55	80	0	135	110	245

Water Production Capabilities

Colour was 3.5 TCU, turbidity was 6.5 NTU and the UV reading was 5.0 HAU. Under these raw water quality conditions total production capacity is 355 ML/d. Water demand for the day was 305 ML and 321 ML were produced. The excess water was placed in the City's reservoirs.

The reservoirs hold a total of 653 ML of water for the purpose of balancing differences in supply from the plants and customer demand. The storage is used to balance short term differences as the demand exerted on the system by customers varies through the day, and to balance longer term differences between the customer demand and plant production. Demand varies randomly whereas plant production can be changed in a controlled manner with the storage acting as a buffer between the two. This prevents variations in water demand from creating undue disturbance to the treatment process by forcing constant changes in throughput and chemical feed rates.

On Saturday, April 1st the reservoirs held 455 ML of balancing storage or 70% of the total. Since run-off was imminent the planned amount was 80% (520 ML), but this could not be achieved due to lost production in the previous week caused by unscheduled shutdowns due to equipment failures.

Sunday April 2nd followed a similar uneventful pattern and by Monday morning the reservoirs stood at 76% (485 ML). Water demand was 316 ML and 319 ML were produced that day.

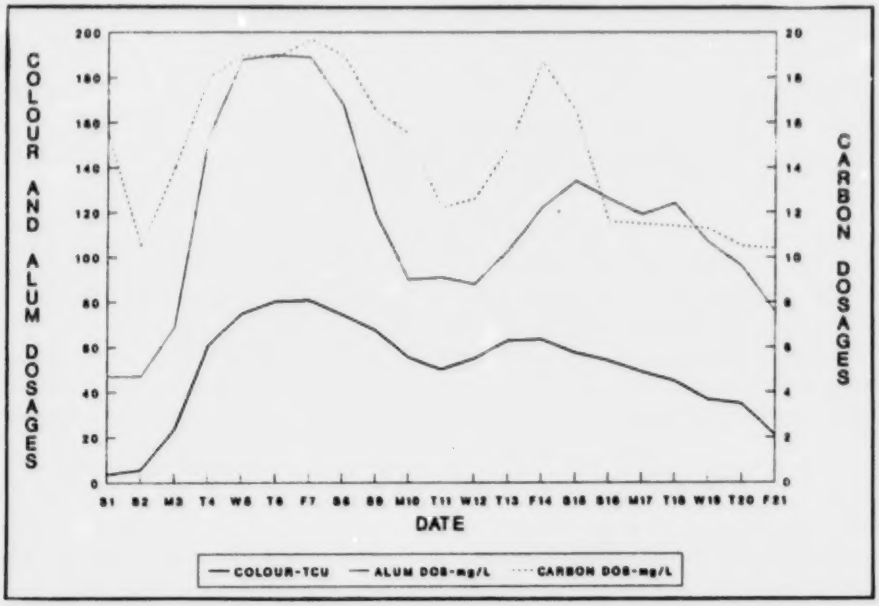
Raw water quality had, however, begun to deteriorate. Colour rose from 8 TCU in the early hours of the day to a peak of 52 TCU. UV followed a similar trend and turbidity increased significantly but did not reach the threshold where it increases the treatability of the water with respect to colour and humics. The results are tabulated below and compared to the quality defined as "Spring Upstream Runoff" in Table 3.

DATE	COLOUR - TCU		UV - HAU	TURBIDITY - NTU
	AVE	PEAK	AVE	AVE
Mon 3	23.5	52	11.8	11
Tue 4	60.5	74	22.5	18
Wed 5	75.0	88	29.0	17
Sp Ups R.O.	90		16	10

The operations staff responded in accordance with the pre-defined strategies (Table 1 and 2). Water production was cut to increase residence time. Laboratory testing for process control and quality control was intensified, particularly with regard to colour, organics and the chloramine and chlorine dioxide disinfection processes. Careful monitoring of the disinfection processes is essential to avoid having chlorine combine with organic molecules to form compounds which have even greater tendencies to cause objectionable tastes and odours. Jar testing was carried out to determine the alum dosage which was then increased accordingly, to approximately 150 mg/L. The dosage of powdered activated carbon was increased to the maximum, i.e. about 18 mg/L. (See Figure 3).

FIGURE 3

Chemical Dosages - April 1989

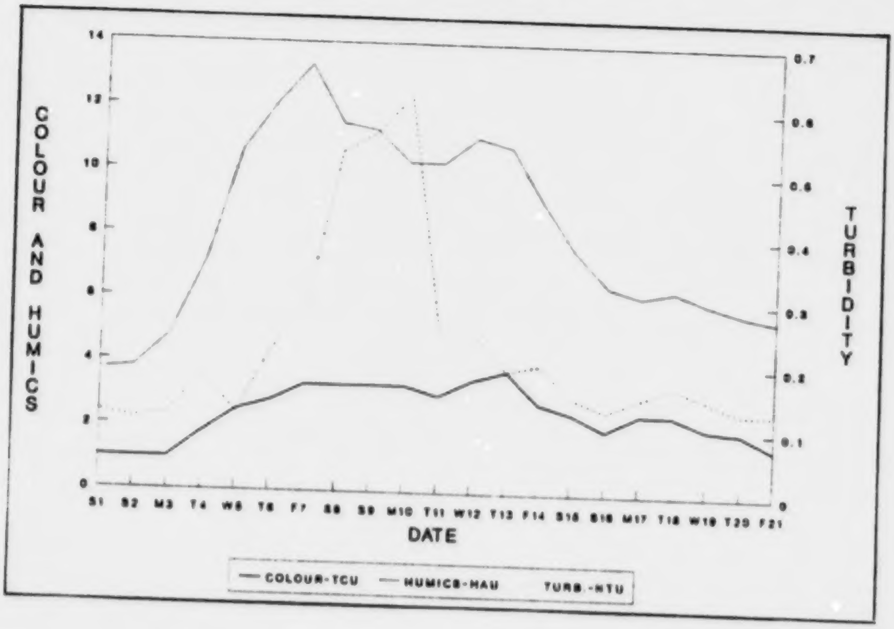


Hence, on Tuesday the 4th of April only 208 ML of water was produced, compared to a demand of 282 ML. At 0700 hours on Wednesday the 5th, the reservoirs stood at 61% (400 ML).

The colour in the raw water was still becoming worse and the alum dosage was again increased to 190 mg/L. Water demand was expected to increase towards the end of the week and the rate of reservoir depletion was unacceptable. The plants managed to increase production to 230 ML/d by Wednesday. Raw water turbidity remained relatively low at around 17 NTU. Because of this the clarification process was becoming less effective as there were not enough dirt particle nuclei for the alum to flocculate well. Alum began to carry over onto the filters, reducing their treatment efficiency and reducing the run time between backwashes. The colour and UV of the treated water deteriorated to give 2.5 TCU and 10.8 HAU daily averages on Wednesday. (Figure 4).

FIGURE 4

Treated Water Quality - April 1989



The same treatment strategies were followed throughout Wednesday, Thursday and Friday in response to worsening raw water quality. Production was increased marginally to 280 ML/d on Friday but this still did not match the demand of 295 ML/d. The reservoirs therefore continued to fall. (See Figures 5, 6).

Alum carry-over to the filters continued to be the major process problem. Treated water quality deteriorated to give the following daily averages on Friday:

Colour	3.3 TCU
UV	13.3 HAU
Turbidity	0.28 NTU

FIGURE 5

Storage Vs Raw Colour - April 1989

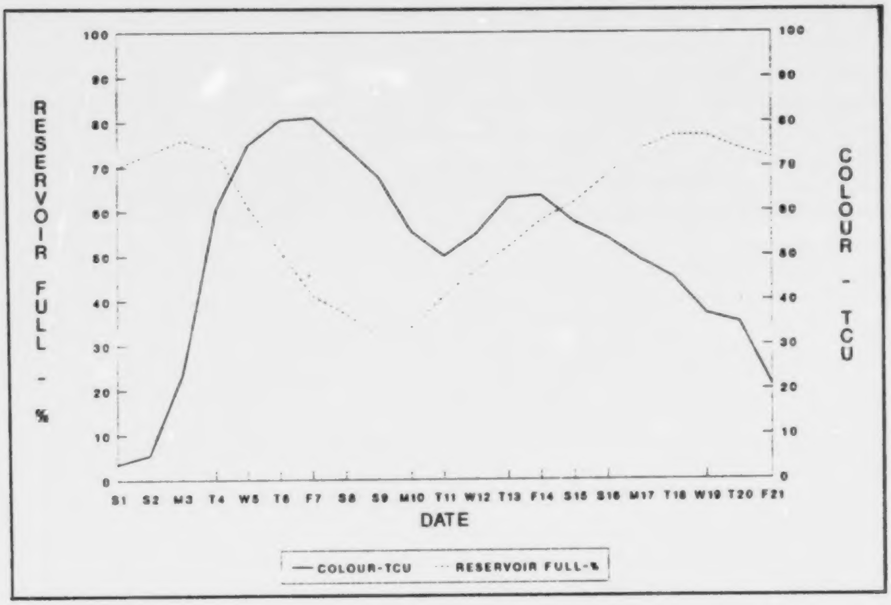
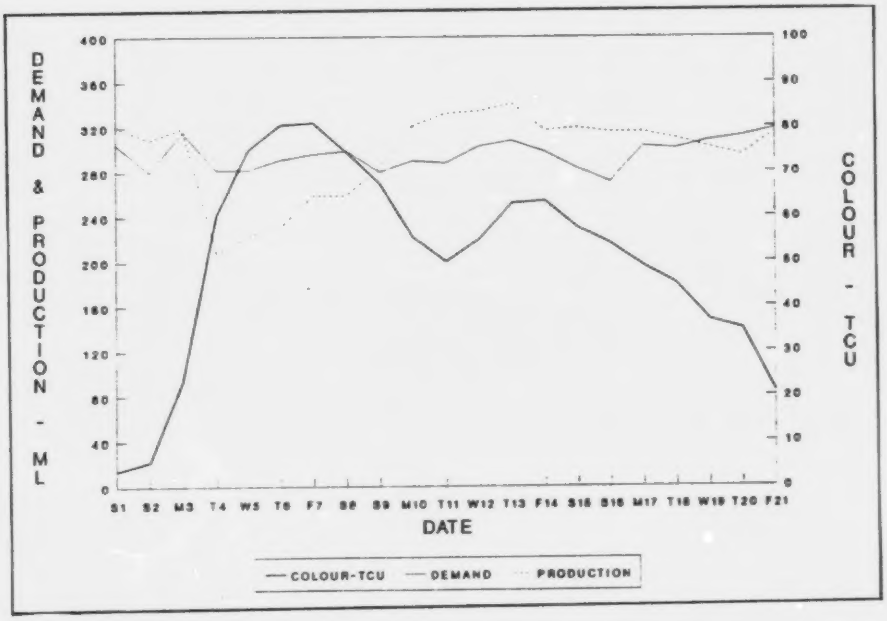


FIGURE 6

Water Production & Demand - April 1989



The City has developed a set of Water Conservation Measures which define a procedure for the staged implementation of demand management methods as reservoirs are depleted.

On Wednesday April 5th, the 0700 hours reservoir level fell below the 65% threshold and Conservation Condition A was implemented.

This basically consists of maintaining supply to all customers, at the low end of the allowable pressure range, together with careful monitoring of pumping and filling operations to ensure that water is placed in storage facilities as strategically as possible. The Regional municipalities which are served by Edmonton Water were notified of the situation with respect to both quality and quantity of treated water.

At 0700 hours on Thursday morning the reservoirs stood at 51% (330 ML) and demand continued to exceed production. Conservation Condition B was implemented. This entails further reductions in pressure, to values a few kPa below alarm points. City departments are instructed to discontinue water use for municipal purposes such as hydrant flushing, street cleaning, fire training, etc. Press releases were issued which explained the situation and requested customers to conserve water. The articles were published in Friday morning's newspapers.

The reservoirs continued to fall and further press releases were issued, together with radio and television news interviews. All media contact was carried out by the department's Public Information staff, leaving operational staff free to dedicate themselves to obtaining the best possible performance from the treatment and distribution system.

Operation of the distribution system becomes complex and difficult when overall storage falls below 30% (195 ML) because each reservoir has different fill and discharge characteristics, together with different patterns of customer demand. 195 ML is a considerable amount of water, but in order to continue to serve the entire City, and Regional customers fairly, water must be available at the correct locations at all times. At 20% overall (130 ML) it was estimated that this balancing act would become impossible, resulting in some districts experiencing total loss of supply. Fortunately, this situation was avoided; the lowest level recorded was 26% (173 ML) at 0001 hours on Sunday the 9th of April.

Throughout Thursday and Friday several pressure complaints were received. Most of these came from high rise residential and office buildings which rely on City pressure to feed the water as high as 8 or 9 floors prior to boosting. Thanks to the public notification most building operators were very understanding. One building installed a new pump to protect them in the future and one maintenance crew found a clogged strainer in a 6 story commercial building, which allowed them to return to normal operations once it was cleared.

From Tuesday to Saturday water demand forecasts were prepared daily and reservoir depletion was updated and projected for the next few days on a graph comparing the trend with raw water colour. (See Figure 5). Each day water transmission staff became more emphatic in their requests for increased production as soon as possible. Each time the conflicting issues of water quality and quantity were discussed a policy was agreed upon, and all parties left the meeting with a commitment to that policy. From Tuesday to Thursday the only quantity issue was the amount of depletion caused by the low production rates required to protect quality. By Friday the questions on quantity were:

(a) What is the lowest reservoir level we can go without causing widespread loss of supply?
(b) When will we get to that point?
(c) How long would it take us to recover?

It was estimated that the lowest acceptable level is around 20%, which could occur late Sunday or sometime on Monday. Once supply to a district was lost it could take a considerable time to get things back to normal because all other storage facilities would be depleted to marginal levels at the same time.

On the morning of Sunday April 9th the reservoirs reached 26% at midnight and recovered to only 32% (207 ML) at 0700 hours. At the operational meeting that morning the management group agreed that water production must be increased to prevent widespread outages, and to maintain full fire protection. Some of the aesthetic parameters of treated water quality would suffer but health aspects would continue to be fully protected at all times. Since treated turbidity was expected to increase, additional bacteriology testing was implemented to ensure that particles in the treated water were not screening bacteria. The test program used the new "Colilert" kits for fast feedback. Alum dosage was cut and, at Rossdale, warm raw water was obtained from the adjacent power plant cooling system, to increase process efficiency. Thanks to some improvement in raw water conditions (colour 67 TCU, UV 24 HAU) and careful operational management treated water quality parameters were kept within reasonable limits:

Colour - 3.3. TCU, UV - 11.3 HAU, Turbidity - 0.56 NTU

On Sunday production only increased to 284 ML because of the time required to make the process changes and move the accumulated alum through the plants. Public response to appeals for careful water use held demands to 280 ML on Sunday and 290 ML on Monday and Tuesday. On Monday 321 ML of water was produced and on Tuesday 330 ML. By Thursday the 13th raw water colour and UV were still high but turbidity had increased to 60 NTU improving treatability. Treated water turbidity was back under control at 0.19 NTU and by Friday the 14th treated colour was below 3 TCU, UV below 10 HAU and turbidity less than 0.2 NTU. Raw water colour and UV continued to decline as turbidity rose. By Friday the 21st raw water conditions could be considered moderate with colour at 21 TCU, UV at 11.6 and turbidity at 60 NTU. At this time the increased production rates had brought the reservoirs back to 72% (470 ML).

The severity and duration of the 1989 run off considerably exceeded the last significant event in 1986 (See Figure 7). The situation was, however, dealt with effectively, without generating any health hazards, or undue levels of complaints regarding water quality and quantity.

The following factors contributed positively to the utility's ability to deliver this performance:

(a) pre-determined procedures for operating the plants to control taste and odour in poor raw water quality conditions, based on effective monitoring of relevant parameters.

(b) a defined strategy for dealing with reservoir depletion over a period of days.

(c) a system of planning production rates and reservoir level management based on demand forecasts.

(d) separate supervisors with responsibility for quantity and quality of treated water.

(e) a senior management willing to conduct daily meetings to mediate conflict of interest on quality and quantity, and to ensure that all staff are committed to the same goals each day. The meetings allowed trends to be considered so that the goals set formed an integral part of a rational plan to deal with the next few days. Each day progress to date was reviewed, the plan updated and the goals set.

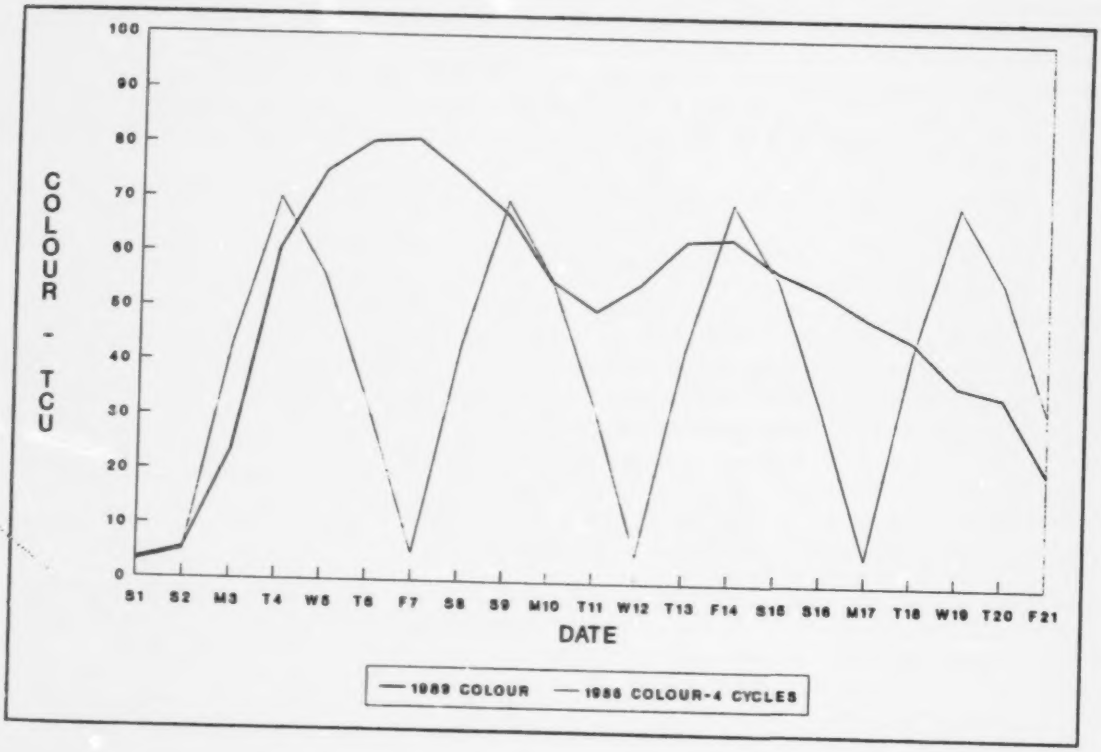
(f) a Public Information Officer who was able to prevent the burden of public contact from falling on Operational Staff, and provide consistency and rationale in a series of media contacts.

(g) a good working relationship based on mutual respect and consideration between water plants staff and major City and Regional customers.

(h) the commitment to water quality and customer service held by all staff and fostered by the corporate culture.

FIGURE 7

Spring Runoff (1989 vs 1988) - April 1989



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2. S.E. Hrudey and Associates. 1986. A Critical Assessment of Drinking Water in Edmonton.

3. I. Rashid. 1987. Ultraviolet Spectrophotometer and Gas Chromatograph as Tools for Monitoring Organics and Controlling Taste and Odour.

DEFINITIONS

1. One Humic Acid Unit (HAU) corresponds to the absorbance required to give a reading on an Ultraviolet (UV) Spectrophotometer at a wavelength of 200 mm equal to the reading given by a solution of standard humic acid at a concentration of 1.00 mg/L.

   The UV reading in HAU's therefore gives an indication of the organic loading as an equivalent concentration of humic acid.

2. Turbidity is the degree of "cloudiness" of water caused by suspended particles ranging from colloidal dimensions to about 0.1 mm. It is measured in Nephelometric Turbidity Units (NTU).

   NTU's are measured by standardized instruments which evaluate the proportion of light scattered by the particles at 90 degrees to the light source.

3. True colour is the colour of water resulting from substances which are totally in solution, with the apparent effects of colloidal or suspended matter eliminated from the measurement. It is measured in True Colour Units (TCU).

   True colour is measured by spectrophotometry in a similar manner to HAU's but at a wavelength of 450 mm.

NOTES

1. All of the data presented in this report is in the form of composite numbers. Raw and treated water quality data area an average of all the results obtained by both Operational and Laboratory staff at both plant sites for each day. Production is the sum total of water produced at all plants and chemical dosages are the total weight of chemicals used at all plants divided by the total throughput. The difference in operational parameters and performance at the separate plants are beyond the scope for this paper.

2. The following details are provided for comparative guidance on treated water quality parameters:

   (a) Treated colour is normally around 1 TCU. During the run-off daily averages rose to a maximum of 3.8 TCU. The aesthetic objective stated in Canadian Drinking Water Guidelines is 15 TCU. It is estimated that customers will see colour in their water at around 8 TCU.

   (b) Treated UV reading is normally less than 4 HAU. We estimate that taste and odour becomes detectable to some customers at about 8 HAU and objectionable to a significant number of people at 10 HAU. This limit was exceeded. Large numbers of complaints were not received, possibly due to public awareness.

   (c) Treated turbidity is normally less than 0.2 NTU. The maximum acceptable concentration in the Canadian Drinking Water Guidelines is 1.0 NTU. This can be allowed to rise to 5.0 NTU as long as effective disinfection is confirmed. Daily averages did not exceed 1.0 NTU but some peaks did, hence the extra bacteria tests to confirm disinfection.

ACCIDENT HISTORY
CITY OF EDMONTON WATER TREATMENT SECTION

G. Johnston
Reid Crowther and Partners Ltd.

INTRODUCTION

Accidents suffered in the work place are a serious concern to both the employee and employer. In 1987 the City of Edmonton Water Treatment Section undertook a study of all accidents, both lost time and no lost time, of which records had been kept. Lost time accidents are those in which the injured employee misses more working time than just the day of the accident. No lost time accidents are those in which the injured employee does not miss any working time other than the day of the accident. This study, which was completed in May 1987, was to analyze accident data to determine if there were any prevalent trends with respect to accident cause, time of day of accident occurrence, etc. which could be brought to light and targeted for future safety programs.

Detailed accident investigation reports for the Water Treatment Section have been kept since January of 1982. Therefore, the data presented in this paper is for accidents which have occurred between January 1982 and the study completion date of May 1987.

The importance of maintaining proper accident reports cannot be overstated. These reports are invaluable in conducting an accident audit such as that completed by the Water Treatment Section. In addition to keeping information on actual accidents, utilities should maintain records on all near miss accidents, those accidents which were narrowly avoided. These reports should contain information on the cause, or potential cause of the accident, and what steps were subsequently taken to ensure that the dangerous situation has been rectified.

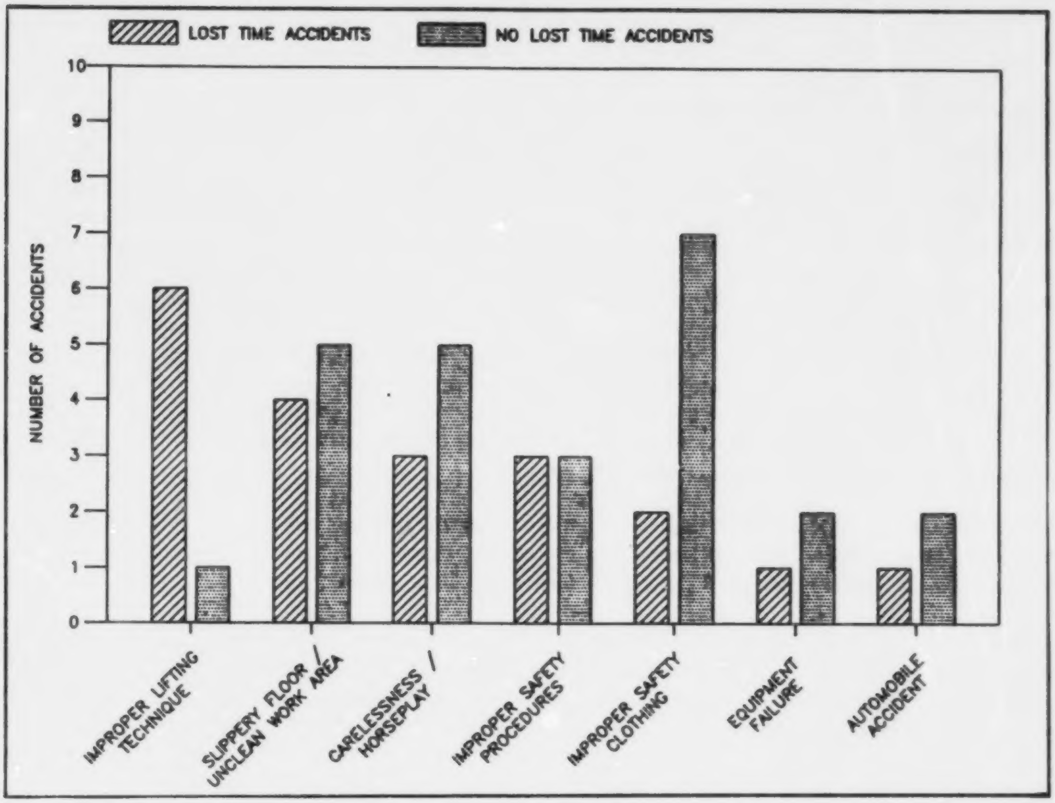
ACCIDENT CAUSE

The 20 lost time accidents and 25 no lost time accidents which occurred during the review period were classified by cause into 7 major groups, as shown in Figure 1. This figure shows that the major cause of lost time accidents was the use of improper lifting technique, causing back injuries. It is interesting to note that accidents caused by improper lifting are of a serious nature, as evidenced by the small fraction of no lost time accidents in this category. In these back injury accidents workers were lifting objects incorrectly, not making use of equipment available to aid in lifting, or simply lifting objects which were too heavy for one person.

The majority of no lost time accidents were caused by employees not wearing proper safety clothing or equipment. An example of an accident included in this category would be an eye injury obtained by not wearing proper eye protection while using compressed air.

Unclean or crowded work areas and carelessness/horseplay were also major accident causes.

FIGURE 1
Accident Cause

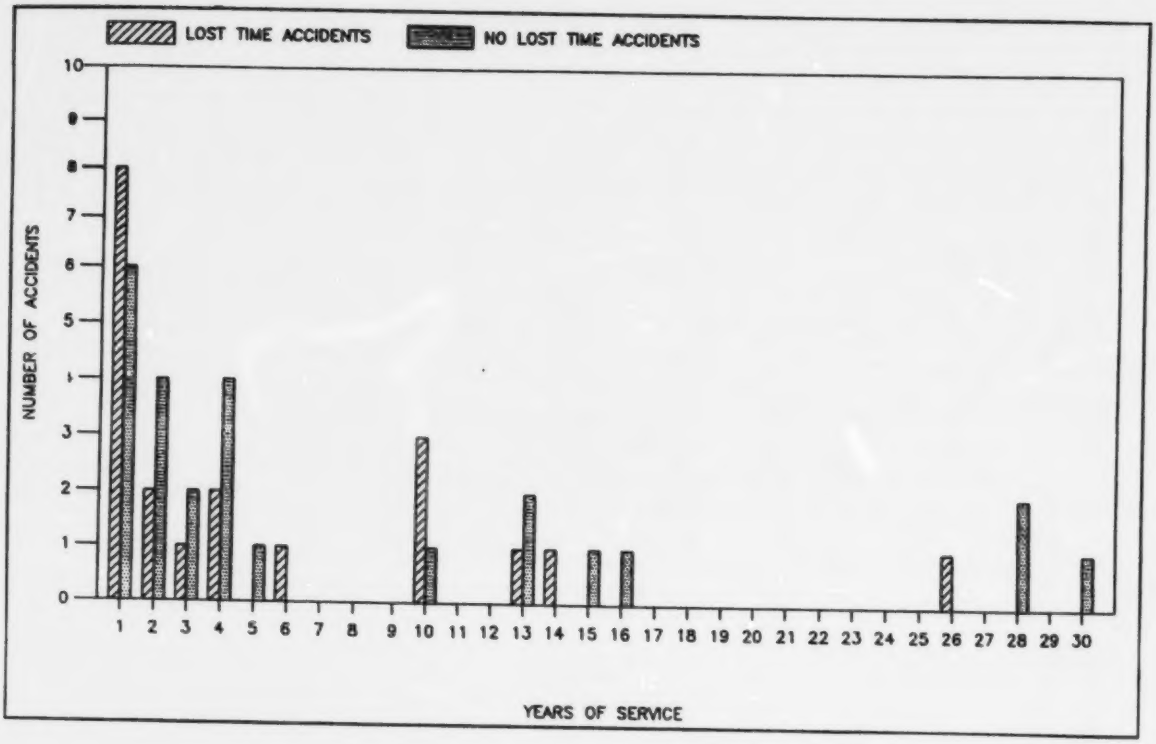


NUMBER OF YEARS SERVICE

Of all lost time accidents on record, 40% occurred to employees with one or less years of service in the Water Treatment Section. This same group was involved in approximately 25% of all no lost time accidents, as shown in Figure 2. The high incidence of accidents to this group is perhaps caused by new employees trying to work quickly in order to impress their supervisors. It may also be attributed to new employees not being completely familiar with proper safety procedures. Of note is the fact that the accidents in which these new employees are involved in are usually of a serious nature, this is illustrated by the high percentage of lost time accidents suffered by this group. That safety is of paramount importance in completing a task should be impressed upon all new employees.

The number of accidents, both lost time and no lost time, generally decreases as the number of years of service increases.

FIGURE 2
Number of Accidents by Years Service



TIME OF DAY AND PERCENTAGE OF SHIFT COMPLETED

This study found that there was a marked increase in the number of accidents which occur right around an employees lunch break; 40 to 50% of shift completed as shown in Figure 3, and between the hours of 11 am and 1 pm, shown in Figure 4. These accidents would be due to employees not fully concentrating on their work, either in anticipation of their lunch break, or by not yet being fully back into their work routine after lunch. One way of getting employees back into a safety frame of mind during this period is to schedule routine safety talks and meetings at these times.

There was another increase in the number of accidents which occurred immediately prior to the end of a shift. Figure 3 shows the relatively large number of accidents which occur when a shift is 100% complete. Rushing to complete the job was a contributing factor in the majority of these accidents. However, in only one case was the injured employee working overtime.

Another interesting observation is that 75% of all lost time accidents occur before noon, as shown in Figure 4. This may be due to the fact that, characteristically, the majority of heavy work is done in the morning, while the afternoon is used primarily for applying the finishing touches to the job and cleaning up. Increased employee supervision during the morning hours may reduce the number of pre-noon accidents.

FIGURE 3

Number of Accidents by Percentage of Shift Completed

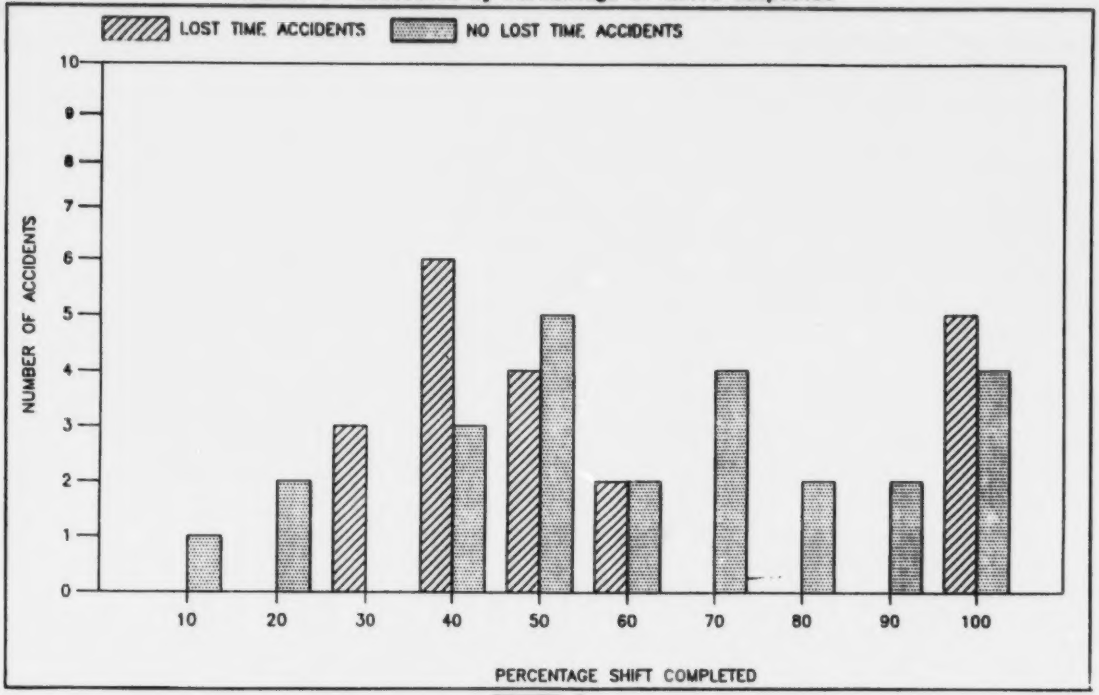
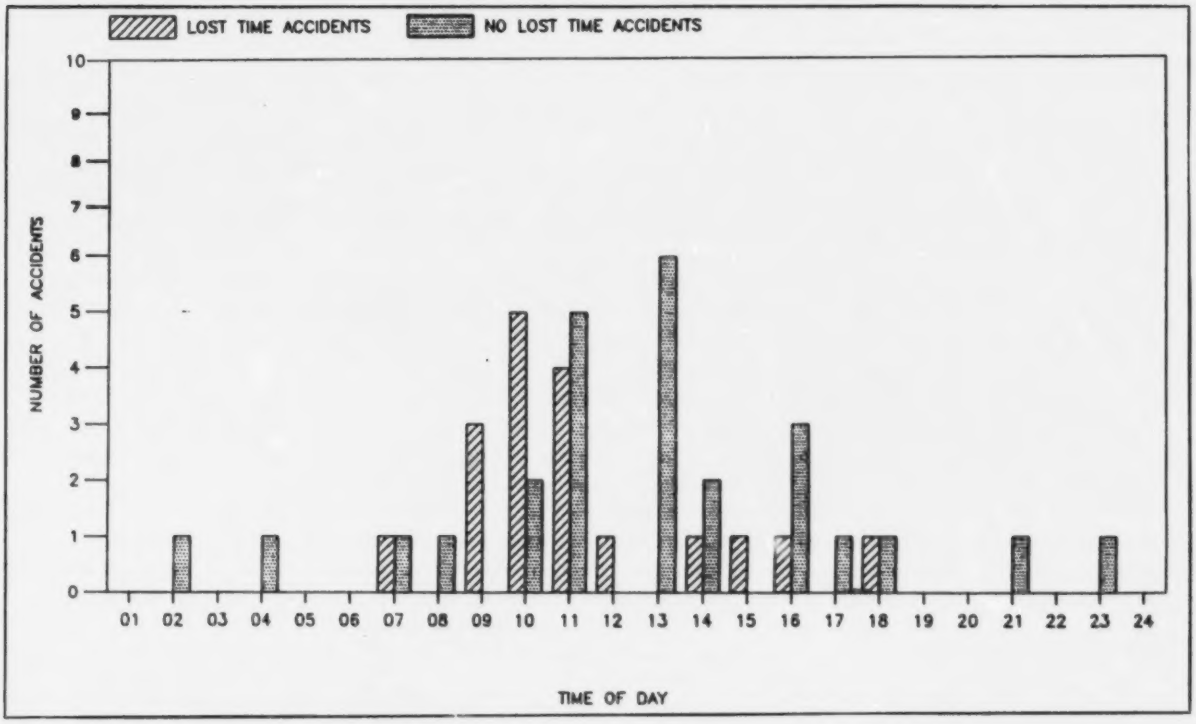


FIGURE 4

Number of Accidents by Time of Day



ACCIDENT FREQUENCY AND SEVERITY

Accident frequency and severity are numerical values which can be used as a measure of accident statistics from time to time. Accident rates for utilities, regardless of size, can be compared using the accident frequency statistic, which reduces the number of accidents to a common denominator and is simply the number of lost time accidents suffered per one million man hours. Normal values for a water treatment plant vary between 10 and 20. An indication of the seriousness of the lost time accidents which have occurred is given by the accident severity, which equals the number of man-days lost due to lost time accidents per one million man hours. Typical values for a water treatment plant vary between 100 and 200.

CONCLUSIONS

While the results presented in this study are specific to the City of Edmonton Water Treatment Section, it is felt that the general trends which are apparent are applicable to other plants as well. As a result of this study, The City of Edmonton Water Treatment Section has launched a campaign against back injuries, the primary cause of lost time accidents. This program has involved proper lifting demonstrations, posters, video tapes and seminars in hopes of raising the employee's level of knowledge with regards to proper lifting techniques and the seriousness of back injuries.

Awareness is a major factor in accident prevention. Raising employee knowledge of the causes and prevalent trends of accidents will increase their awareness and help to reduce the number of accidents in the future.

BANFF TOWNSITE SEWAGE TREATMENT FACILITY

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INTRODUCTION

The Banff Townsite is located in Banff National Park in the Canadian Rockies. The permanent population presently numbers about 10,000 people although the total population fluctuates according to the number of visitors.

Sewage from the townsite is collected in an underground gravity collection system and conveyed to the present sewage treatment plant site south of the community in close proximity to the east end of the Banff Springs Golf Course. Treated effluent is charged to the Bow River.

The present sewage treatment plant was constructed in 1970. Influent is coarse screened and comminuted prior to entering a 30 ML aerated lagoon. Lagoon effluent enters a small polishing pond and is chlorinated prior to discharge. This facility was designed to treat a very dilute sewage with an influent BOD of 30 mg/L to 60 mg/L.

Since the original plant was designed, the Banff Townsite has experienced substantial growth, both in permanent resident housing and in visitor accommodation facilities.

During the same period, substantial upgrading of the sewage collection system was undertaken to limit extraneous flows. Consequently, the sewage flows increased substantially in strength although only marginally in quantity. Concurrently, federal government standards for effluent from federal establishments were tightened. Eightieth percentile standards for BOD and TSS were set at 20 mg/l and 25 mg/l, respectively.

By the late 1970's, the present plant was exhibiting problems in treating the more concentrated sewage to the degree necessary to satisfy the new standards. In response to these difficulties, Parks Canada commissioned a feasibility study in 1982 to consider the treatment system and recommend a course of action which would alleviate present and future substandard discharges to the Bow River (UMA, 1983).

In 1985, Reid Crowther was engaged to finalize the design concept and prepare final design drawings and specifications. The sewage treatment plant went to tender in May 1988 and began construction in July, 1988. Construction is now underway and should be completed by August, 1989 at an approximate cost of $10.3 M.

During evaluation the design concept, several process alternatives were considered, including:

Grit Removal

- Vortex grit chambers
- Aerated grit chambers

Primary Treatment
- Fine screens
- Primary clarifiers

Secondary Treatment
- Rotating Biological contactors
- Activated sludge - Conventional
- Activated sludge - Sequencing Batch Reactors
- Extended Aeration - Conventional
- Extended Aeration - Sequencing Batch Reactors
- Aerated Lagoons

Disinfection
- Chlorination
- Ultra-violet Light

Sludge Thickening
- Gravity Thickening
- Centrifuges
- Dissolved Air Flotation
- Gravity Belt Thickening

Sludge Digestion
- Aerobic Digestion
- Anaerobic Digestion
- Autoheated Thermophilic Aerobic Digestion (ATAD)

Sludge Dewatering
- Sludge Lagoons
- Belt Filter Press
- Centrifuges

Through the evaluation, Parks Canada Environmental Protection Service (EPS) and Wastewater Technology Centre (WTC) staff provided review and input.

This paper will summarize many of the considerations accounted for in the selection of the sewage treatment processes incorporated in the Banff Townsite Sewage Treatment Facility and will describe several unique features of the design.

SEWAGE CHARACTERISTICS

The design flows and loads were based on population figures, flow measuring and sampling carried out in 1988, projected to the year 2003. Flows were estimated for various seasons and for wet and dry weather periods. The design flow was based on the maximum month anticipated for the design year. Table 1 summarizes the pertinent design flows and loads.

TABLE 1
Projected Flows and Loads

Design Year	2003
Basis for Flows	Maximum Month
Average Flow	15.5 ML/d
Average Dry Weather Flow (ADWF)	14.0 ML/d
Average Wet Weather Flow	20.5 ML/d
Peak Dry Weather FLow (PDWF)	25.5 ML/d
Peak Wet Weather Flow (PWWF)	32.0 ML/d
Average Total BOD Load	2325 kgBOD/d
Sewage Characteristics	
BOD	150 mg/L
TSS	175 mg/L
Effluent Criteria - Average	
BOD	15 mg/L
TSS	20 mg/L

PROCESS DESCRIPTION

Figure 1 shows the plant layout and Figure 2 illustrates the process schematic. Table 2 summarizes the basis design parameters for the major process components included in the plant design.

FIGURE 1
Banff Sewage Treatment Facility - Plant Layout

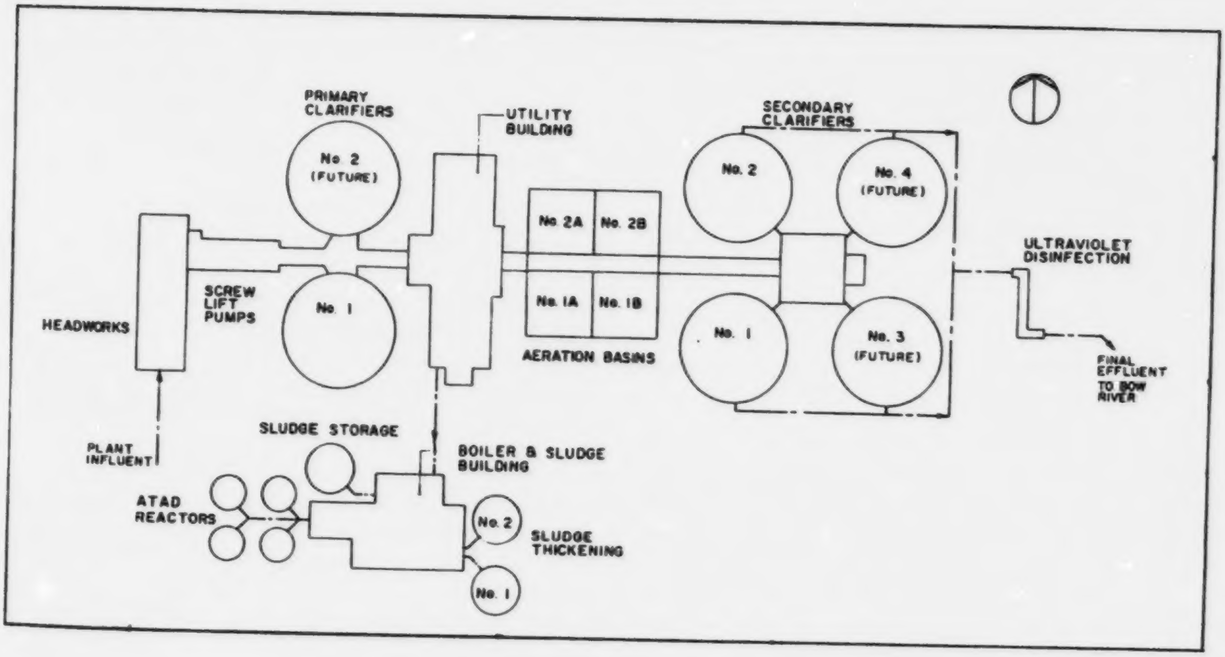
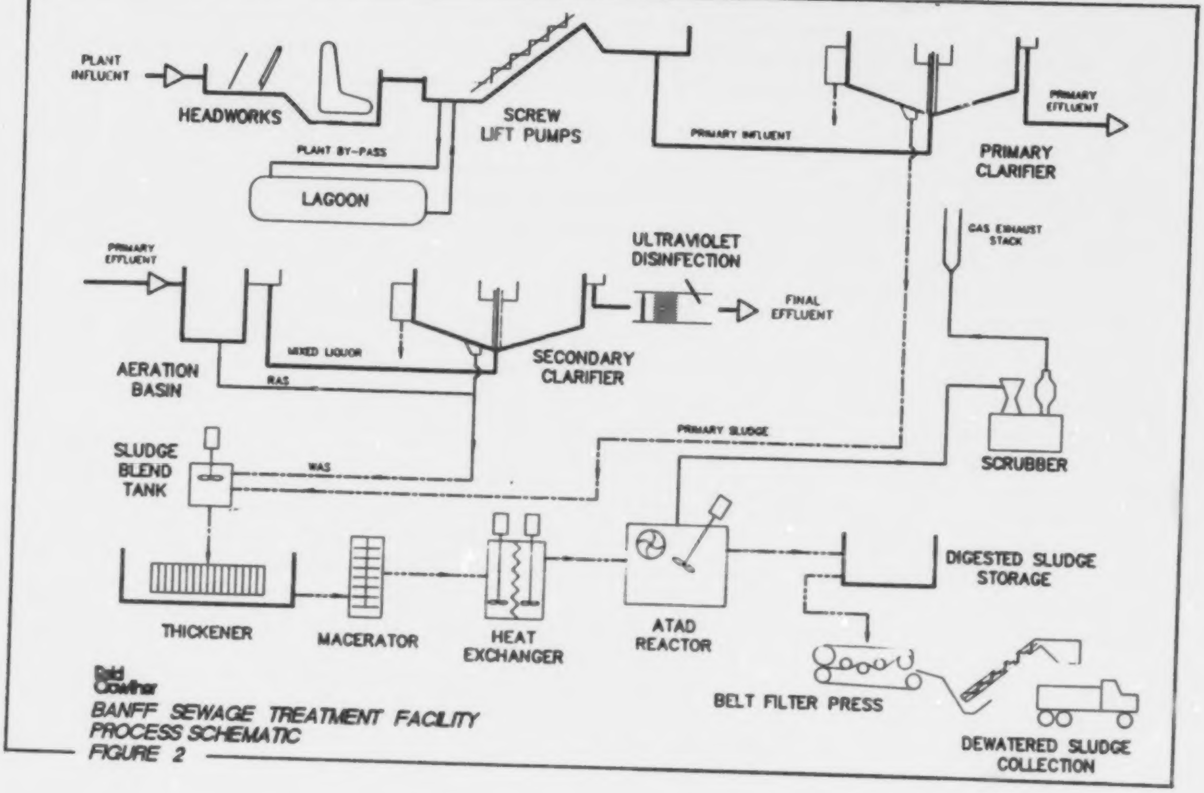

PRIMARY CLARIFIERS
UTILITY BUILDING
SECONDARY CLARIFIERS
No. 2 (FUTURE)
No. 2A
No. 2B
No. 2
No. 4 (FUTURE)
ULTRAVIOLET DISINFECTION
SCREW LIFT PUMPS
HEADWORKS
No. 1
No. 1A
No. 1B
AERATION BASINS
No. 1
No. 3 (FUTURE)
FINAL EFFLUENT TO BOW RIVER
PLANT INFLUENT
SLUDGE STORAGE
BOILER & SLUDGE BUILDING
ATAD REACTORS
No. 2
SLUDGE THICKENING
No. 1

TABLE 2

Summary of Design Parameters

PRELIMINARY TREATMENT	
Coarse Screen	75 mm openings
Medium Screen	20 mm openings, mechanically raked
Grit Removal	2 min. retention time, aerated @ 0.3 m^3/min./m
Grit & Screening Collection	4 day storage, 2.0 m^3 container
Equalization	30 ML (existing lagoon)
Raw Sewage Pumping	3 screw lift pumps @ 16 ML/d
PRIMARY TREATMENT	
Primary Clarifier	1 unit @ 18.3 m diameter Average O/F rate = 54 m^3/m^2d
Primary Sludge Pumping	Single Diaphragm Air-Operated Pump
SECONDARY TREATMENT	
Aeration Basins	2 trains of 2 basins SRT = 5 days HRT = 4 days MLSS = 2800 mg/L
Aeration	SOTR = 2160 kg O_2/d Air Supply = 81.1 sm^3/min.
Secondary Clarifiers	2 @ 18.3 m diameter Average)/F Rate = 27 m^3/m^2d
RAS Pumps	2 @ 7000 m^3/d, variable speed Centrifugal Screw-Impeller
DISINFECTION	
Ultraviolet Radiation	Number of Lamps - 288 Number of Banks - 4
SLUDGE THICKENING	
Blend Tank	Nominal Retention Time - 3 hrs. Volume - 20 m^3
WRc Gravity Thickeners	2 units @ 6.4 m diameter
Thickened Sludge Pumping	2 peristaltic pumps @ 10 L/S
DIGESTION	
ATAD	4 reactors HRT = 6.5 days Aeration Power Input = 110 w/m^3
Sludge Storage	Nominal Retention Time - 5 d
DEWATERING	
Belt Filter Press	Feed Rate - 11.0 m^3/hr.
Polymer Feed	5 mg/kg

FIGURE 2
Banff Sewage Treatment Facility - Process Schematic


PLANT INFLUENT
HEADWORKS
SCREW LIFT PUMPS
PLANT BY-PASS
LAGOON
PRIMARY INFLUENT
PRIMARY EFFLUENT
PRIMARY CLARIFIER
GAS EXHAUST STACK
ULTRAVIOLET DISINFECTION
PRIMARY EFFLUENT
FINAL EFFLUENT
AERATION BASIN
MIXED LIQUOR
RAS
SECONDARY CLARIFIER
PRIMARY SLUDGE
SLUDGE BLEND TANK
WAS
SCRUBBER
DIGESTED SLUDGE STORAGE
THICKENER
MACERATOR
HEAT EXCHANGER
ATAD REACTOR
BELT FILTER PRESS
DEWATERED SLUDGE COLLECTION
Reid Crowther
BANFF SEWAGE TREATMENT FACILITY
PROCESS SCHEMATIC
FIGURE 2

The following paragraphs will summarize many of the considerations addressed in the development of this process scheme.

Preliminary Treatment, Flow Equalization and Raw Sewage Pumping

The plant influent flow passes through a coarse screen, a medium screen and an aerated grit channel. Medium screenings and grit which are removed from the flow are deposited on a conveyor for transport to a waste container. The waste screenings and grit periodically will be removed from the plant and hauled to Banff's solid waste handing facility. The preliminary treatment process units are enclosed in the Headworks Building.

Immediately downstream of Preliminary Treatment, the flow enters the sump of a screw lift pump station. Three screw lift pumps each sized at 50% of the peak flow (16 ML/d) raise the sewage about 5.0 metres. The treatment plant site is underlain by coarse granular deposits. The groundwater table in this strata is influenced by the Bow River and rises quickly during the spring thaw. Founding the downstream process units below the water table would have required significant expenditures for dewatering during construction and for the structural components required to ensure flotation did not occur. Consequently, raw sewage pumping offered a long term savings for this facility.

Immediately upstream of the screw lift pumps, provision has been made for the bypass of peak flows exceeding the maximum day wet weather flow of 25.5 ML/d. The bypassed peak flows will be directed to the existing lagoon. This bypass allowed for the optimal sizing of downstream treatment units. The lagoon aeration system will be modified to allow for its continued use in order than an odour problem does not develop. When the influent flows recede, the bypassed sewage will be returned to the plant.

In the evaluation of process alternatives, continued use of the existing aerated lagoon to perform some function was a major consideration. Possible uses which were reviewed included pre-aeration, effluent polishing, digested sludge storage, and the ultimate choice - flow equalization. This option was the most cost effective, without adversely affecting treatment process flexibility or reliability.

Primary Treatment

One circular primary clarifier was incorporated in the plant design to remove particulate suspended solids and oil and grease. An air operated single diaphragm pump has been provided for primary sludge pumping.

Secondary Treatment

Conventional activated sludge was chosen for the plant due to cost, flexibility and reliability considerations. Two process trains have been provided, each with two completely mixed basins in series. Primary effluent can be fed to the first of the two basins, or step fed to the two basins.

Oxygen is provided by a coarse/medium bubble aeration system. The air supply to each basis is controlled to maintain design D.O. values. Inlet valves on the centrifugal blowers, which supply air to the system, are controlled to maintain a constant discharge pressure.

Two circular secondary clarifiers, one for each process train, have been provided for solid/liquid separation. Each clarifier is equipped with a flocculation centre well in which gentle mixing is provided. These centre wells allow for the re-flocculation of activated sludge particles and for the future process addition of chemical phosphorous removal.

Sludge is removed to the central removal well by a rotating scraper mechanism. Siphon sludge removal was not incorporated in the design. Full width skimming has been included. The effluent launder was designed with a horizontal baffle extending about 0.6 metres into the basin. Others (Stukenberg et al, 1983) have found this technique effective in limiting solids carry over due to density currents along the outer wall. Separate secondary sludge pumps are supplied for each of the two process trains to allow for the segregation of flow. Return Activated Sludge Pumping is accomplished using variable speed, screw impeller centrifugal pumps. The pumping rate is paced by the influent flow. Waste Activated Sludge pumping is done with constant speed, screw impeller centrifugal pumps. These pumps will operate at set intervals for a period controlled according to an algorithm based on the solids retention time (SRT).

Disinfection

Secondary effluent will be disinfected prior to discharge using ultra-violet light. UV was chosen over chlorination to reduce the potential hazard inherent in transporting gaseous chlorine through the Banff Townsite and in handing the gas on site. It also allows for the removal of chlorine residuals from the discharge to the Bow River where they could prove harmful to aquatic life.

Sludge Thickening

Adequate sludge thickening is critical to downstream sludge stabilization processes, as will be discussed in a later section. Several process options were considered for sludge thickening, including the following:

Co-thickening in the primary clarifiers
Gravity thickening
Dissolved Air Flotation
Centrifugation
Gravity Belt Thickening

Co-thickening of waste activated sludge (WAS) and primary sludge in primary clarifiers is simple and can be used to accomplish sludge densities in excess of 5%. However, the overflow rates used for design must be lowered to provide for this process option, thereby increasing clarifier size. Also, two units should be installed to ensure some process reliability. There is an increased potential for odour generation, exacerbated by the large water surface area in primary clarifiers. In addition, these units are subject to hydraulic surges due to fluctuations in the influent flow which in turn can impact sludge settlement and thickening. Nitrified sludges are especially difficult to co-thicken as the large quantity of substrate and the active biomass avail themselves to denitrification.

Primary sludge withdrawal from primary clarifiers can be controlled to obtain sludge thicknesses between 3% and 5%. As this is the require consistency for downstream process units, separate WAS thickening can be used to obtain this concentration for the other sludge stream. The two sludges then may be blended prior to stabilization. This type of configuration was considered for Dissolved Air Flotation (DAF), centrifugation, and gravity belt-thickening.

DAF is relatively simple and reliable once it is set up properly and when it is operated under constant load. Hydraulic fluctuations caused by intermittent WAS pumping adversely affect performance. Operating problems can be caused by plugging nozzles and failures in skimming mechanisms. Odours can be generated by the stripping of volatile compounds from solution. Operating costs are higher than gravity thickening or co-thickening due to the power requirements for gas charging and due to polymer costs. The complexity of the system, the potential for odour generation, the adverse impact of intermittent loads, and the relatively high capital and operating cost are disadvantages of a DAF system.

Centrifugation for thickening of sludges suffers from many of the same problems as DAF. It requires less space and is less prone to odour generation, but it is more sensitive to intermittent and/or varying loads and for the size of unit required for Banff, it is more expensive.

Performance of gravity belt thickening is not as susceptible to variations in sludge flow and is relatively simple to operate. The major drawback is the polymer requirement. To obtain 4% WAS concentrations, dosages up to 8.0 mg/kg TSS would be required.

Gravity thickening of WAS generally is not very effective. The low density of WAS particles renders them difficult to settle. Intermittent WAS pumping creates hydraulic surges which cause solids carryover which return to the main process train. Denitrification in the thickener of a biological sludge causes rising sludge. Control algorithms relating sludge blanket depth to pumping rate and frequency generally are not employed to optimize process performance. However, gravity thickening does have some advantages. It is a simple process compared to other mechanical sludge thickening processes. The equipment is similar to that used for other sewage treatment processes and so readily is understood by operating staff. Polymer and power usage are minimal.

For these reasons, a modified version of sludge thickening as developed by the Water Research Centre (WRc, Great Britain), was investigated for use in Banff. Based on a favorable technology evaluation and cost comparison, the WRc gravity thickener was incorporated in the Banff design.

The WRc thickener was developed in the United Kingdom to replace existing process units which were designed mostly as sludge storage chambers. Thickening in these units was a secondary concern, both in design and in operation. Consequently, thickening efficiencies were minimal and as plants approached their design capacity, the sludge processing units often became hydraulically overloaded. This problem was evident especially in small to medium sized plants where the more complex thickening technologies may not be applicable. Coupled to this was the trend in the UK to reducing staffing levels at sewage treatment plants. A higher degree of automation with a coincident increase in process effectiveness was required.

WRc investigated these problems and developed a thickener design compatible with the dual goals of increased process efficiency and decreased operator attention. In doing so, an understanding of the thickening properties of biological sludges was obtained.

The three main features of a WRc thickener which differ from conventional design are:

Full depth picket agitation
Flat bottom design
Peripheral sludge drawoff
Sludge blanket control

The full depth pickets provide gentle mixing and allow trapped gases to escape. The thickeners have flat bottoms and peripheral sludge drawoffs to simplify construction. As the sludge mass tends to rotate with the pickets, the central core becomes more dilute due to centrifugal forces. The most dense part of the blanket was found to be about two-thirds of the tank radius out from the centre. Thus the peripheral drawoff was considered a more suitable location for removing sludge and the costs associated with a sloped bottom were not considered justified.

Control of the sludge blanket depth was the most important concept. By using and maintaining infrared blanket sensors, sludge drawoff is controlled to maintain optimal blanket height.

For the Banff plant, tow of the WRc thickeners were provided. Primary and secondary sludges are mixed in a short-term (< 6 hours) blend tank and fed on a relatively continuous basis to each unit. Sludge is withdrawn by peristaltic pumps. Feed and withdrawal rates are controlled within WRc parameters to obtain adequate performance.

Digestion

Three sludge digestion processes were investigated for the Banff Townsite Sewage Treatment Facility: Conventional Aerobic Digestion, High Rate Anaerobic Digestion, and Autoheated Thermophilic Aerobic Digestion (ATAD). The latter process option, ATAD, was chosen for the plant due to the following:

- The large reactor capacity and high power input required by conventional aerobic digestion made this alternative uneconomic.

- When compared to ATAD, high rate anaerobic digestion was more costly in terms of capital layout but required less for operation and maintenance. Overall, the costs were similar. Thus, due to the risk inherent in handling the corrosive and hazardous gases generated in anaerobic digestion and the attendant need for a flare in a national park setting, high rate anaerobic digestion was disregarded.

- ATAD has proven successful in West Germany as a simple method of sludge digestion. In addition, the level of sludge disinfection attained would render the stabilized sludge more suitable for land application in the Park. ATAD is a modification of aerobic digestion in which the heat generated by the exothermic oxidation of sludge organics is conserved, consequently raising the temperature to the thermophilic range (about 55 deg.C to 60 deg.C). Heat generation and conservation is a primary concern. For this reasons, the following are important:

  - The sludge must have a TSS content over 3% and a VSS content over 2%.

  - The reactor must be insulated to minimize heat loss from the reactor floor, walls, and roof.

  - An efficient aeration device must be used to minimize heat stripping.

Thermophilic digestion is self-limiting as the reaction begins to slow at temperatures over 62-63 deg.C. However, if close to these temperatures are obtained, the volatilization of ammonia increases substantially, possibly leading to odour problems. Hence, an upper limit of 5% TSS has been suggested to limit the potential for this occurrence. An aspirating aeration device has been specified for the Banff project. In West Germany, these have been used with success in more than 50 ATAD installations, 150 mm of insulation will be applied to the reactor and 100 mm to interconnecting piping.

Thickened sludge is pumped from the thickeners to a holding tank/heat exchanger. This unit allows sludge to be fed to the digesters on an intermittent basis and also recaptures some of the heat from digested sludge. Intermittent feeding increases pathogen kills in the process with negligible effect on performance.

Digested sludge is pumped from the "heat exchanger" to a storage basin where it is held for dewatering.

Dewatering

A belt filter press was chosen for sludge dewatering on the basis of least cost and compatibility with the chosen sludge disposal option. Digested sludge is fed to the unit, polymer added, and the dewatered cake conveyed to a trailer. Dewatering will be required every three to five days, depending on plant loading.

Composting

Dewatered sludge will be hauled to a paved area approximately 100 metres from the plant. There it will be mixed with wood chips and composted material in the proportions necessary to obtain the correct nutrient ratios and moisture content. The blended material will be windrowed on the site and periodically mixed using windrow mixing equipment. Finished compost will be stockpiled on the site. It is anticipated that the compost will be used in the park where soil amendments are necessary to reclaim disturbed areas.

ACKNOWLEDGEMENTS

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LAGOONS IN COLD CLIMATES AND THEIR PERFORMANCE IN SASKATCHEWAN

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INTRODUCTION

Lagoons have been accepted as simple and economical wastewater treatment systems which are particularly suitable for small communities. In general, the advantages of lagoons are as follows:

1. low capital cost;
2. low operational cost;
3. controlled discharge capacity;
4. ability to provide reasonable treatment;
5. flexibility to weather conditions;
6. simple to construct and easy to maintain; and
7. low energy requirements.

Lagoons have been used for wastewater treatment from tropics to the arctic, but each climate exerts its own special effects which must be considered in the design and operation of such facilities. Climatic conditions exhibit a marked effect on waste stabilization pond operation in Canadian prairies. Winters are long and often severe, while the summers are short and snow remains for extended periods of time. Ice cover on the Prairies generally last from mid-November through the end of April. Ice thickness reaches a maximum of 1 m on the ponds. Snow cover in lagoons up to 1.37 m and 0.9 m have been reported in Manitoba and Saskatchewan respectively (1). Temperature not only affects the biological and physical-chemical processes, but influences ice cover, water density and mixing and distribution patterns of the biological population. The factors affecting lagoon performance in cold climatic conditions including the following:

1. temperature;
2. microbial kinetic rate;
3. ice and snow cover;
4. thermal stratification;
5. wind;
6. organic loading rate;
7. number and shape of cell; and
8. discharge control measures.

During periods of relatively warm weather and adequate sunlight, the overall treatment performance is generally good. During cold weather and limited sunlight, the treatment performance is usually poor. This variability of the treatment performance necessitated a number of design and operational modifications to overcome the limitations of the process. This paper reviews the information on the design, operation of wastewater lagoons in Canada and analyzes the performance of lagoons in the Province of Saskatchewan.

There are about 1000 wastewater lagoons in Canada. This represents approximately 50 percent of the wastewater treatment units reported in Canada. Alberta, Manitoba, Saskatchewan, the Northwest and Yukon Territories rely heavily (70 percent or more) on this form of treatment (2). Oswald and Gotaas (3) classified ponds according to the biological reactions as aerobic, anaerobic and facultative. The National Inventory of Municipal Waterworks and Wastewater Systems in Canada (4) indicated that in Saskatchewan out of 242 wastewater treatment facilities 188 facilities use the lagoon as classified: anaerobic 8; aerated 13; and facultative 168. This figure does not include the unsurveyed rural areas. Considering all hamlets, settlements, etc. the total number of lagoons in Saskatchewan grouped under different types, exceeds 450 as shown in Table 1(5).

TABLE 1

Types of Lagoon Systems in Saskatchewan

SNo.	Type of System	No. of Systems
1.	Single-cell anaerobic lagoons	5
2.	Anaerobic lagoons with storage	11
3.	Single cell facultative lagoons	11
4.	Two cell facultative lagoons	58
5.	Multi-cell faculative lagoons	377
6.	Aerated lagoons	9
7.	Aerated Lagoons with storage	6
	Total number of systems	477

Effluent Quality Guidelines and Lagoon Design Standards

Information regarding waste stabilization pond practices in Saskatchewan and Canada during the 1960's are available in the literature (1,6,7). Effluent quality in terms of 5 day biochemical oxygen demand (BOD) in Canadian wastewater lagoons ranges from 10 mg/L to 30 mg/L. The suspended solids (SS) level ranges from 10 mg/L to no limit conditions (8). The Water Pollution Control and Waterworks Regulations of Saskatchewan (9) subsection 38(1) stipulate that:

> "Treatment facilities shall be operated to provide an effluent that meets the requirements set by the minister"

Therefore, in Saskatchewan the effluent quality criteria for discharging any waste is site specific and will have to be determined on the basis of individual requirements. Fisher et al (1) indicated that many of the ponds in the Province of Saskatchewan discharge onto or across farmland and receive little or no dilution; consequently, specific effluent standards have not been established. Ponds are designed to discharge the effluent in the late fall prior to freeze up and again in the spring when dilution from runoff is available.

The two common criteria used in the design of lagoons are: (1) organic loading; and (2) retention time. Typical design requirements of Canadian Provinces are summarized in Table 2. Some designers consider the main question as the maximum permissible organic loading to achieve the desired effluent quality without creating unacceptable odour conditions. Odour associated with the operation of waste stabilization ponds are common in cold climate lagoons. A short period of fairly strong odours usually accompanies the spring turn over from complete ice cover to open water conditions due to biological activity and release of produced gases. Oleszkiewicz and Sparling (10) have indicated that the maximum loadings of 55 kg BOD/ha.d used in Manitoba for facultative lagoons be reduced to 35 kg BOD/ha.d, in order to avoid odour nuisance during the spring breakup. A study on the anaerobic short detention period ponds in the Province of Alberta concluded that serious odour problems are not necessarily associated with the poor degree of wastewater treatment (1). The same authors also indicated that extensive overloading of facultative ponds in Saskatchewan had resulted in many cases in extending the spring odour period well into the summer months. It was further stated that water supplies having more than 400 mg/L of sulfate concentrations in Alberta and Saskatchewan have created more odour problems in stabilization ponds. Excessive seepage and evaporation and exposure of wastewater mound have also been reported to cause odour related problems. Information on the design aspects of cold climate wastewater lagoons are available in the literature (2,8,11,12,13,14,15,16,17,18,19,20). The US EPA (12) design manual tabulates design based on conventional approaches and indicates that an organic loading of 11 to 22 kg BOD/ha.d is acceptable when sizing a lagoon for regions where ice cover forms. Environment Canada (8) mentioned that EPA's guidelines had failed to meet effluent standards under continuous flow conditions in cold region lagoons. Canter et al (14) reported that typical BOD loadings rates across the United States demonstrated a trend towards reduced loading and longer detention times at higher latitudes. The study indicated a maximum loading rate of 22 kg BOD/ha.d for regions with seasonal ice cover. Dawson and Grainage (13) reported on the design criteria for wastewater lagoons in arctic and sub-arctic regions and suggested 22 kg BOD/ha.d as a criterion for northern Canada.

For Saskatchewan, the Water Pollution Control and Waterworks Regulations (9) subsection 13(3) stipulate that:

> "Unless otherwise approved, all facultative lagoons shall be designed to have:
>
> (a) a minimum of two basins operating in series;
>
> (b) primary basins with a surface area sufficient to ensure that a BOD loading of no greater than 30 kilograms per hectare per day will be applied; and
>
> (c) a combined storage capacity in all basins other than the primary basins, of at least 180 days of sewage flow into the basins for the service connections and population to be served by the sewage works."

In addition, for summer-type operations for which 75 percent of the wastewater flow occurs from May to September, the primary cell may be sized on the basis of BOD loadings of 55 kg/ha.d (21).

The foregoing information indicates that each facility will have to be designed on the basis of local conditions. Some factors that influence this decision are the location of the facility, location of the water table, the type of soil, and thermal conditions, etc.

Lagoon systems come in a variety of forms, configurations and discharge modes. Different cells have different functions to perform. A study on the performance of northern regions wastewater lagoons indicated that winter BOD and solids removal efficiencies are not significantly improved by longer detention times in a single cell, but are significantly improved when a system has several cells operated in series. The BOD removal efficiencies are not affected by an increase in lagoon depth (22). Beier (23) concluded that for once-a-year discharge lagoons in Alberta, one or more facultative cells did not provide as good a treatment as two or four anaerobic cells with two or more treatment/storage cells.

TABLE 2
Typical Provincial Design Criteria
for Wastewater Lagoons (8)

Province	Loading	Effluent Quality mg/L	Detention Time	Other
Nova Scotia	20 lb BOD_5/acre-day (22.4 kg/ha.d) 100 gal/cap.day (454 L/cap.d) 0.17 lb BOD_5/cap.day (0.18 kg/ha.d)	BOD 10 stream SS 10 BOD 30 ocean SS 30	60 days	chlorination
Prince Edward Island	35 lb BOD_5/acre-day (39.2 kg/ha.d) 200 people/acre (81 people/ha) 60 gal/cap.day (272.4 L/cap.d)	BOD 20 22 20 provided 10:1 dilution available	120 days	nil
New Brunswick	34 lb. BOD_5/acre-day (38.1 kg/ha.d) 200 people/acre (81 people/ha) 0.17 lb BOD_5/cap.day (0.19 kg/ha.d)	BOD 20 SS 20 provided 8:1 dilution available	60 days	case specific
Ontario	24 lb BOD_5/acre-day (26.9 kg/ha.d)	BOD 15 SS 20	approx. 365 days	phosphorus removal to 1 mg/L
Manitoba	50 lb BOD_5/acre-day (56.1 kg/ha.d) 200 people/acre (81 people/ha)	BOD 30 SS no limit	186 day operating capacity	nil
Saskatchewan	30 lb BOD_5/acre-day (33.6 kg/ha.d) (primary cell)	N/A	2 cells secondary cell 180 days	cells in series
Alberta	50 gal/cap.day (227 L/cap.d)	BOD 15-30 SS 15-100	365 days ND 3 years evaporation pond	nil

Dawson [24,25] reported that stabilization lagoons operating under extreme winter conditions provide BOD and solids removal rates that are approximately equivalent to primary treatment. Lagoon geometry is found to influence the rate of dispersion in the lagoons which consequently controls the complete-mix conditions. Middlebrooks et al. [11] mentioned that rectangular tanks have a length to width ratio of 1:1 to 2:1 would provide the best plug flow conditions. Discharge of lagoon effluent can be continuous, intermittent or non-existent (controlled and total retention). Continuous discharge lagoons require less land and operating time than controlled discharge lagoons. Lagoons are operated in parallel especially in winter for distributing organic loadings. For best operation the lagoon can function in series all summer and in parallel during winter [2]. The Water Pollution Control and Waterworks Regulation [9] Subsection 13(4) of Saskatchewan stipulate the following for lagoon discharges:

> "Unless otherwise approved, no owner or operator of a lagoon shall cause or allow any discharge from the lagoon in the period commencing on November 1 in any year and ending on the spring run off in the following year as determined by the minister".

Under normal operating conditions, a two-cell lagoon is discharged twice a year; (1) during spring run off and (2) during fall before November 1 [26].

It can be seen from Table 1 that about 60 percent of lagoon systems in Saskatchewan are of multi-cell facultative type. The primary anaerobic cells are placed ahead of the main treatment cell to reduce the loading on the main treatment cell, and to provide flexibility in the accumulation and removal of settled solids as sludge. An anaerobic lagoon treating dilute wastewater over 9 winter months removed 30 to 40 percent BOD, 50 percent SS and 70 percent coliforms [2]. In general, anaerobic cells in cold climates are expected to provide BOD and SS removals equivalent to primary treatment, and are designed on the basis of retention times between 2 to 5 days, with depth ranging from 3 to 6 m. The minimum and maximum depth of 0.6 and 1.5 m respectively is stipulated for primary cells in Saskatchewan [21].

Main treatment cells are of facultative type comprising one or more cells in series and parallel having 1.5 to 2 m deep. Storage ponds or polishing ponds have anywhere between 6 to 12 months retention - even 2 years in the high arctic - with BOD loadings ranging from 7 to 29 kg BOD/ha.d [2].

Saskatchewan Environment [27] stipulate that secondary lagoon cell storage capacity should be of the order of 210-230 days of wastewater flow for effluent irrigation.

Lagoon Performance in Saskatchewan

The data used for the analyses were collected from Saskatchewan Environment and Public Safety from various lagoon effluent discharge particulars for three years 1985, 1986 and 1987 and from a literature source [4]. The parameters used for analyzing lagoon performance were BOD, SS, total Kjeldahl-nitrogen (TKN), total phosphorus (P) and total coliforms. The data were analyzed for 5 facilities having single-cell anaerobic lagoons, 20 facilities having two-cell facultative (part storage) lagoons and 27 facilities having multi-cell facultative lagoons.

The characteristics of lagoon effluents vary with time. The presentation of data from the analyses should recognize the various exhibited by the lagoon effluent and display them. Usually only average values are furnished, but it is desirable to indicate the range values. Many of these lagoon effluent characteristics can be transformed to approximate normal distribution. These can be displayed as a straight line on arithmetic probability paper. Certain types of data such as bacterial density are logarithmically normal and will plot as a straight line on log probability paper. Thus the distribution of the values for each parameter can be examined for randomness. If the data are randomly distributed, the data will produce a normally distributed curve. The details of such procedures are available in the literature (28,29,30).

The arithmetic probability plot in respect of BOD and SS for three types of lagoons investigated for the years 1985, 1986 and 1987 are presented in Figures 1 through 8.

FIGURE 1

Probability Plot for Effluent BOD
in Single-Cell Anaerobic Lagoons

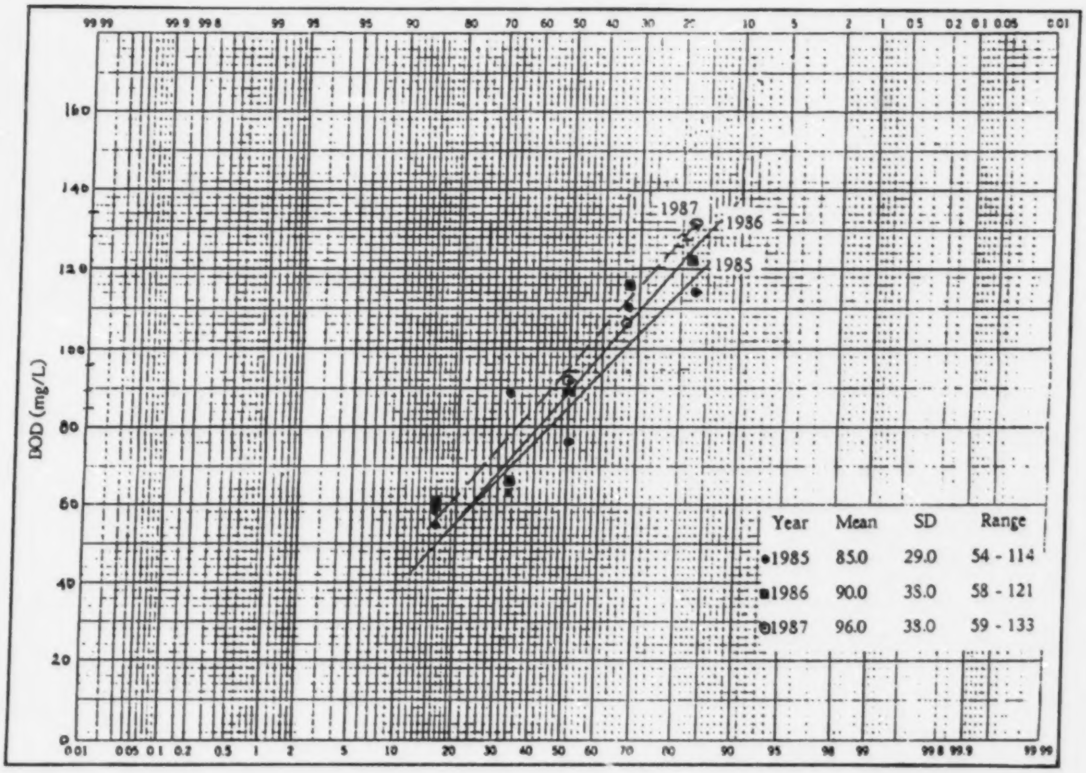


FIGURE 2

Probability Plot for Effluent SS in Single-Cell Anaerobic Lagoons

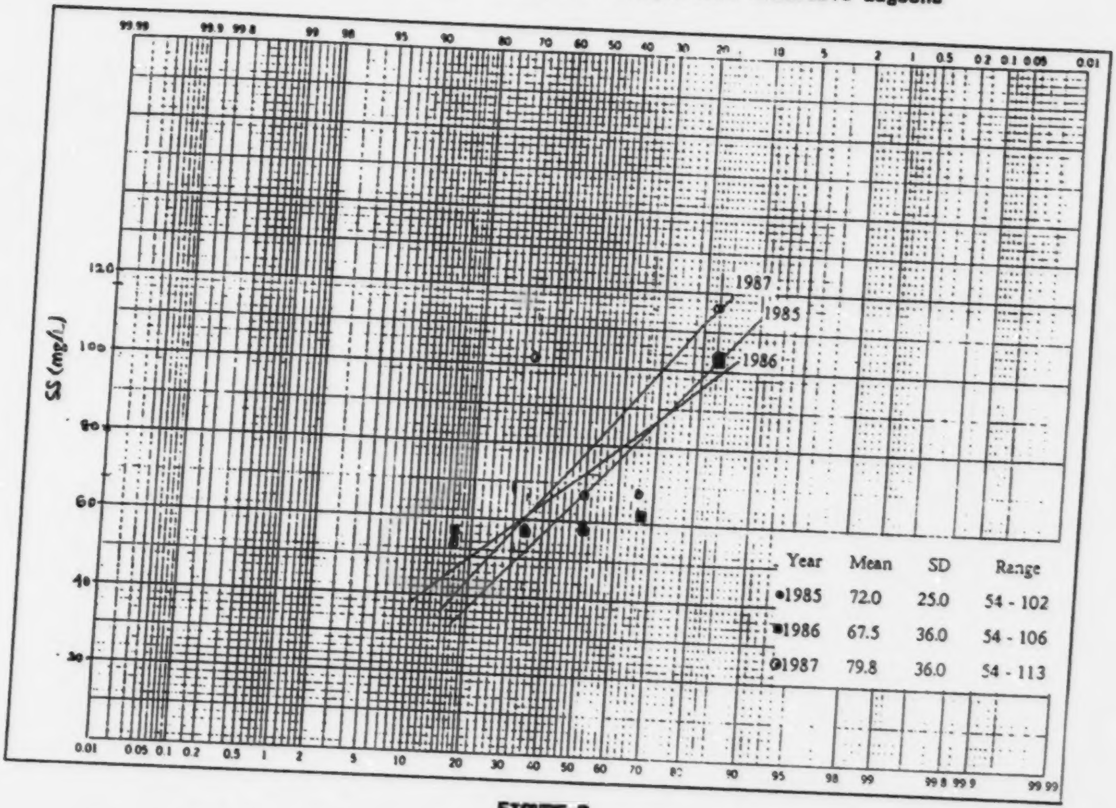


FIGURE 3

Probability Plot for Effluent BOD in Two-Cell Facultative Lagoons

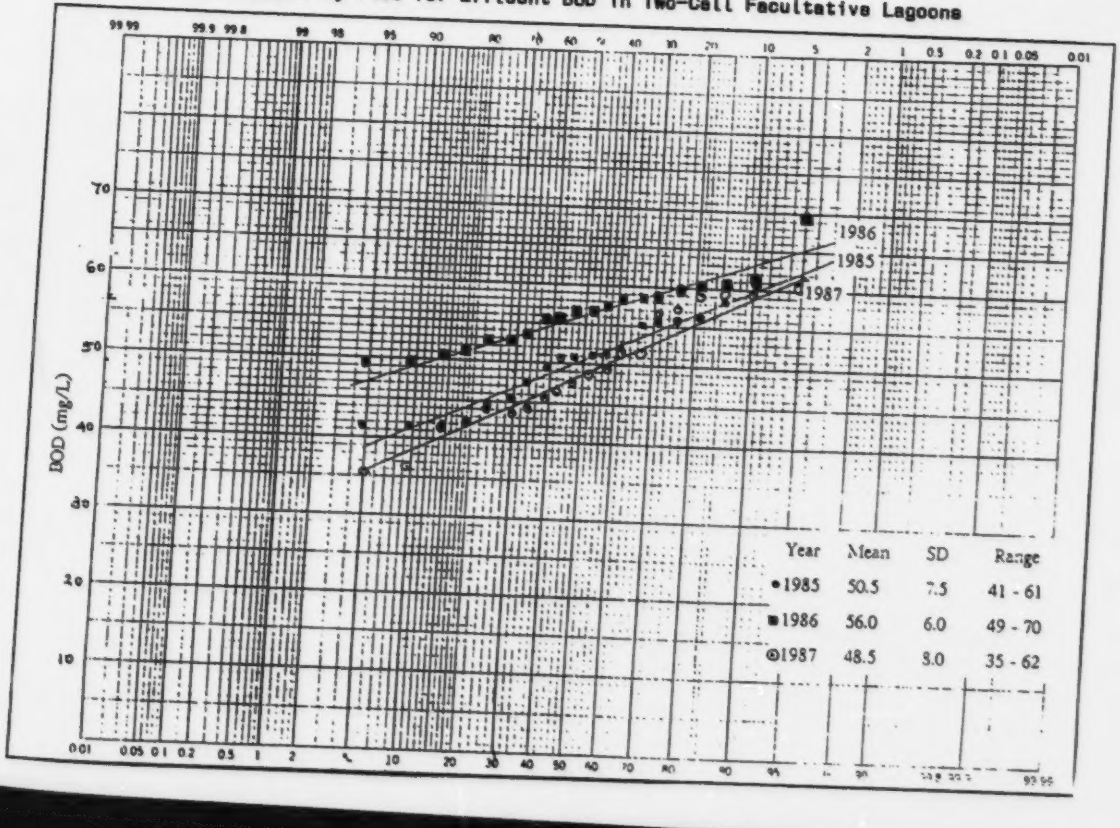


FIGURE 4
Probability Plot for Effluent SS in Two-Cell Facultative Lagoons

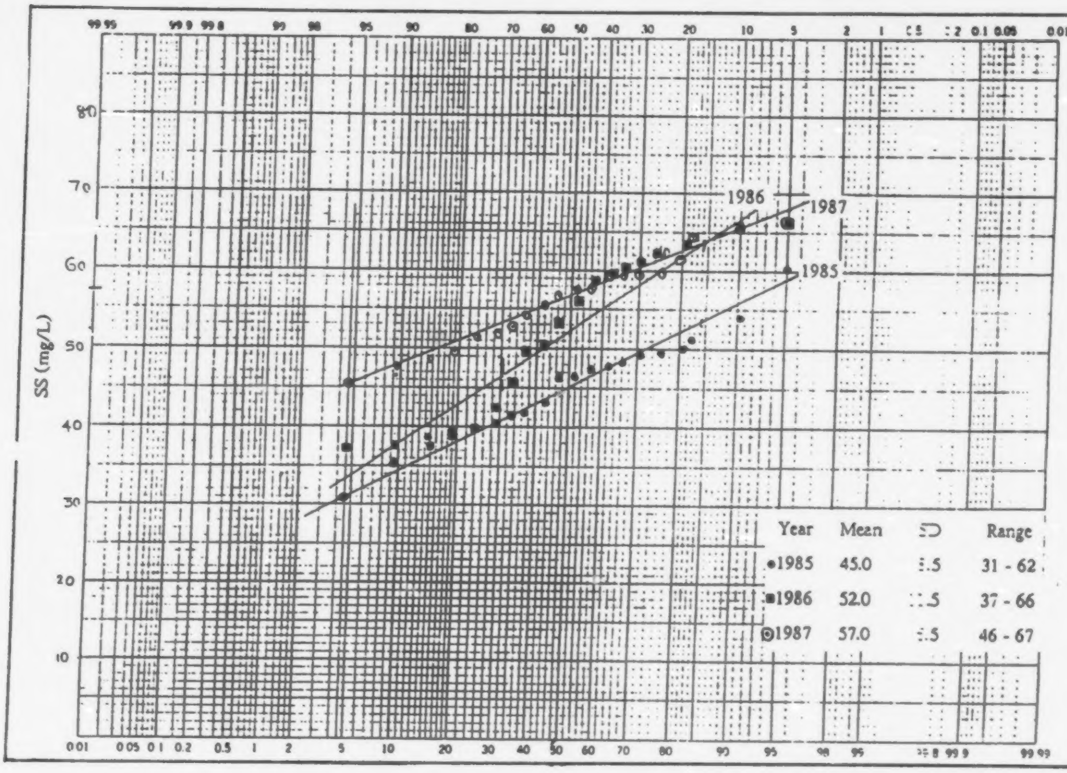


FIGURE 5
Probability Plot for Effluent BOD in Multi-Cell Facultative Lagoons (for Spring Discharge)

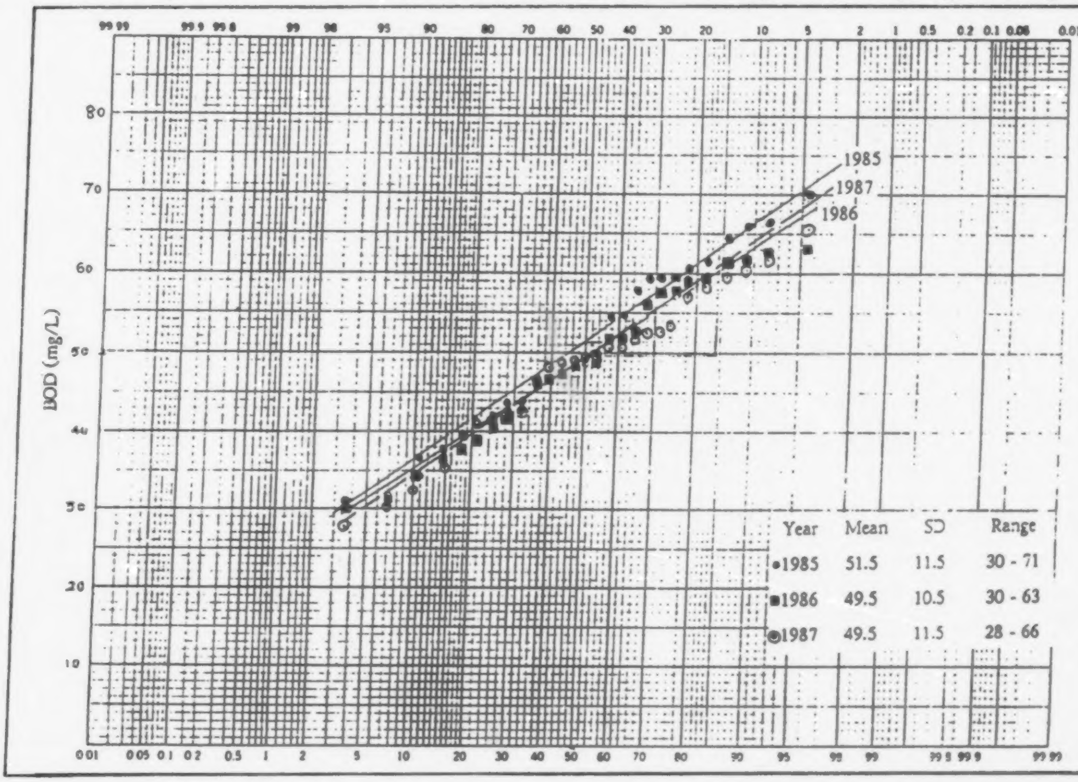


FIGURE 6

Probability Plot for Effluent SS in Multi-Cell Facultative Lagoons (for Spring Discharge)

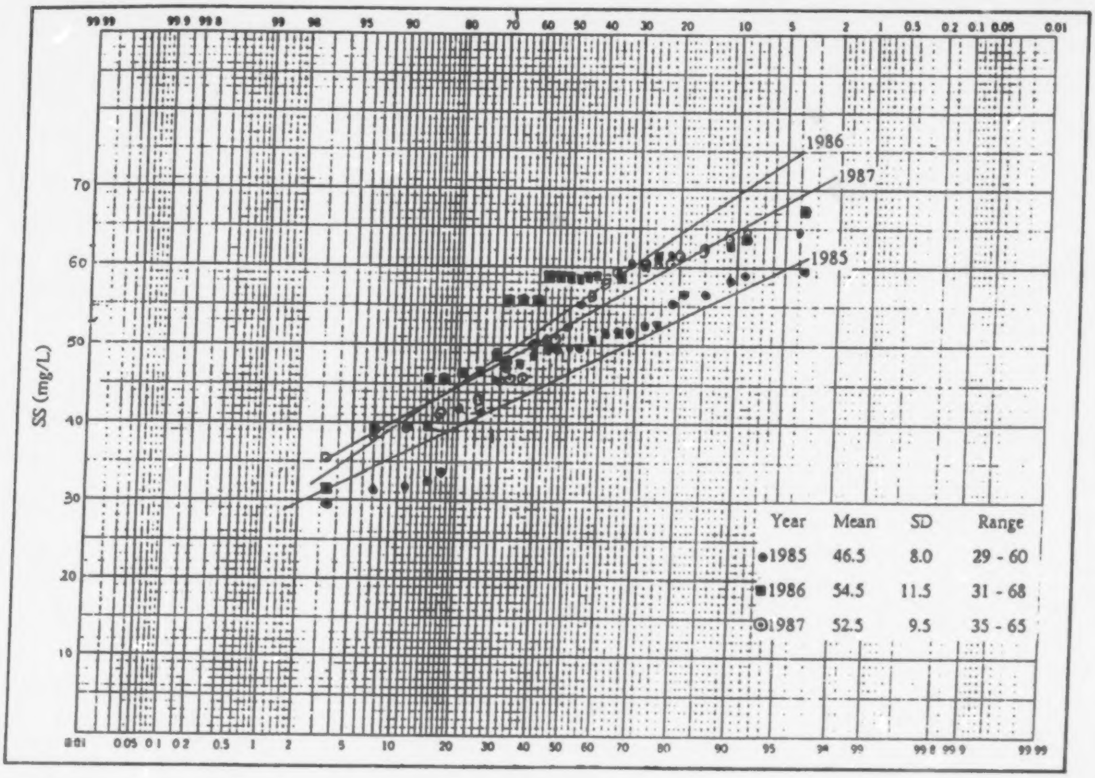


FIGURE 7

Probability Plot for Effluent BOD in Multi-Cell Facultative Lagoons (for Fall Discharge)

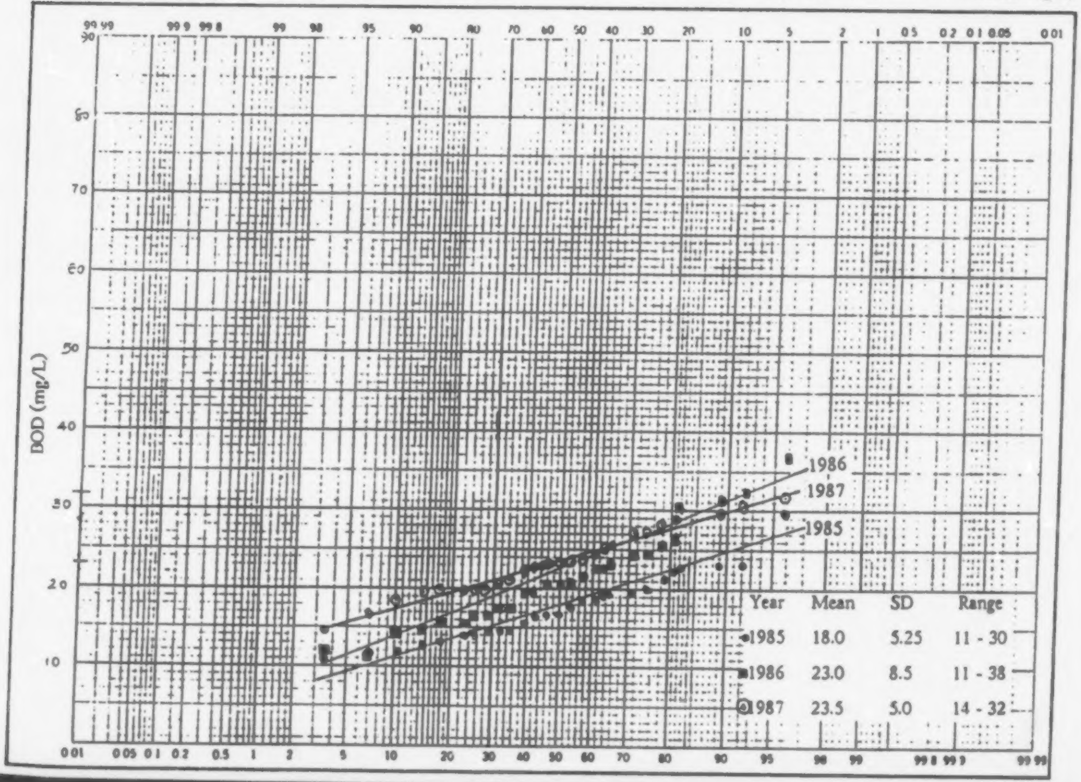
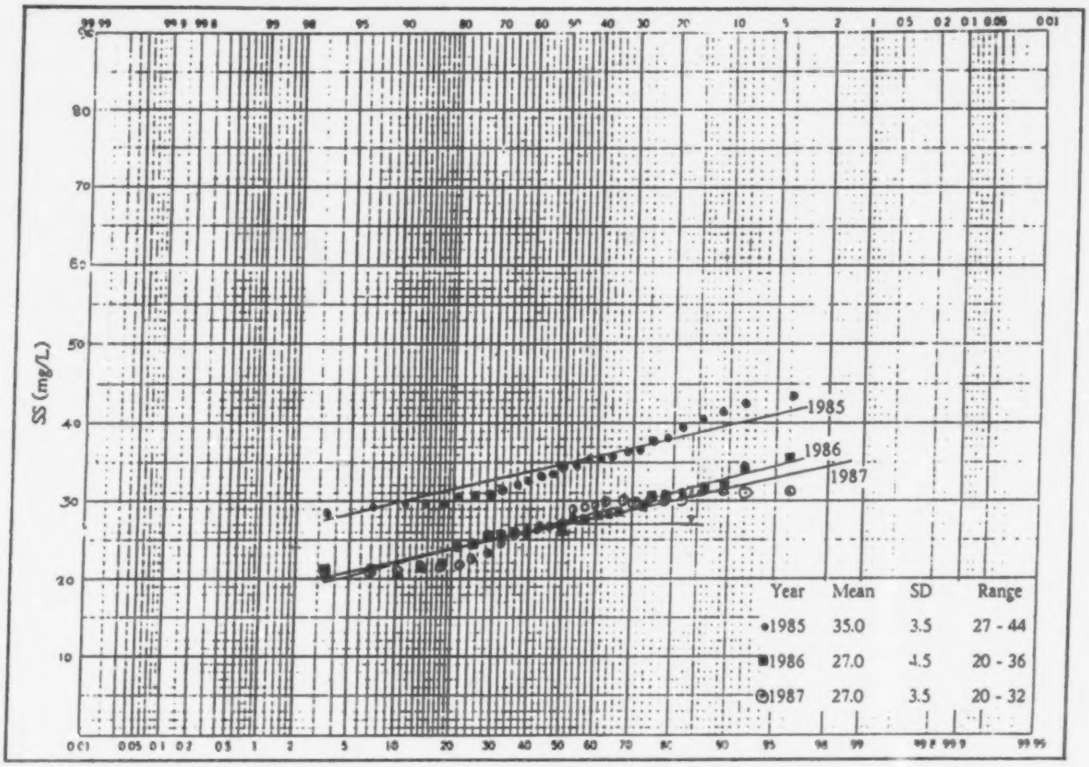


FIGURE 8

Probability Plot for Effluent SS in Multi-Cell Facultative Lagoons (for Fall Discharge)



The average design flow for these facilities ranged from 100 m^3/d to 2800 m^3/d. The raw wastewater characteristics are shown below:

Characteristics	Range (mg/L)	Typical Value (mg/L)
BOD	80 - 280	180
SS	60 - 250	155
Total Phosphorus	5 - 15	10
TKN	5 - 55	30
Total Coliforms	1.8 - > 2.5 x 10^6 per 100 mL	

Results and Discussion

The details of average effluent concentrations for BOD, SS, TKN, P and coliforms for the three types of lagoon systems studied for the years 1985, 1986 and 1987 are presented in Table 3 to 6.

TABLE 3

Details of Effluent Concentration for Single-Cell Anaerobic Lagoons

PARAMETER	YEAR					
	1985		1986		1987	
	Mean	SD	Mean	SD	Mean	SD
BOD	85.0	29.5	90.0	38.0	96.0	38.0
SS	72.0	25.0	67.5	36.0	79.8	38.0
TKN	34.0	9.0	36.0	7.0	35.5	7.5
P	6.1	1.0	6.6	1.1	5.8	0.8
Total Coliforms /100 mL	$> 1.7 \times 10^6$		$> 2.5 \times 10^6$		$> 1.8 \times 10^6$	

TABLE 4

Details of Effluent Concentration for Two-Cell Facultative Lagoons

PARAMETER	YEAR					
	1985		1986		1987	
	Mean	SD	Mean	SD	Mean	SD
BOD	50.5	7.5	56.0	6.0	48.5	8.0
SS	45.0	8.5	52.0	11.5	57.0	8.5
TKN	28.1	7.0	33.5	7.0	22.5	6.0
P	5.5	1.0	7.0	1.3	6.4	1.8
Total Coliform /100 mL	$< 1.9 \times 10^6$		$< 2.1 \times 10^6$		$< 1.6 \times 10^6$	

TABLE 5

Details of Effluent Concentration for Multi-Cell Facultative Lagoons for Spring Discharge

PARAMETER	YEAR					
	1985		1986		1987	
	Mean	SD	Mean	SD	Mean	SD
BOD	51.5	11.5	49.5	10.5	49.5	11.5
SS	46.5	8.0	54.5	11.5	52.5	9.5
TKN	36.5	6.0	25.0	5.0	32.0	7.0
P	5.15	1.25	5.0	1.4	5.3	1.0
Total Coliform /100 mL	$< 1.0 \times 10^5$		$< 1.5 \times 10^5$		$< 0.9 \times 10^5$	

TABLE 8

Details of Effluent Concentration for Multi-Cell Facultative Lagoons for Fall Discharge

PARAMETER	YEAR					
	1985		1986		1987	
	Mean	SD	Mean	SD	Mean	SD
BOD	18.0	5.25	23.0	8.5	23.5	5.0
SS	35.0	3.5	27.0	4.5	27.0	3.5
TKN	18.5	3.5	13.5	4.0	18.0	4.0
P	4.2	0.7	3.5	0.8	4.7	0.9
Total Coliform /100 mL	< 3.3×10^4		< 4.2×10^4		< 5.5×10^4	

In Saskatchewan wastewater lagoons are considered as primary and/or secondary treatment facilities. In the absence of specific wastewater effluent discharge guidelines, no comparisons can be made regarding meeting the effluent standards by all these facilities. In general, all the three types of lagoon systems studied showed an average effluent BOD ranging from 17.5 mg/L to 93.5 mg/L, with multi-cell facultative lagoon performing much better. The performance with reference to SS, P and TKN followed the same trend.

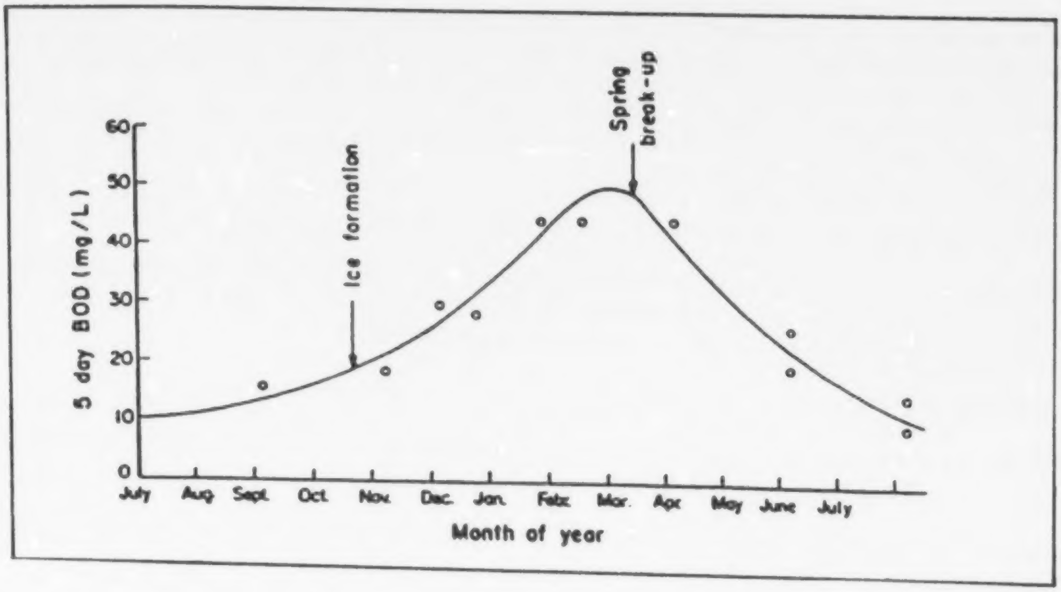
BOD Removal

BOD reductions based on typical values of raw wastewater ranging from 73 to 90 percent were achieved in the multi-cell facultative lagoon. Further the lagoon effluents were of excellent quality during the fall discharge (90 percent reduction), while spring discharges exhibited comparatively low quality (73 percent reduction). Beier (23) while evaluating the lagoon performance in Alberta reported that once-a-year fall discharge BOD value averaged 8 mg/L as against the twice-a-year fall discharge value of 14 mg/L.

McDonald (7) reported that in a waste stabilization pond in Moose Jaw with two anaerobic cells and two aerobic cells the BOD removals ranged from 22 to 32 percent in winter; when only two aerobic cells were operated during summer (due to odour problems the anaerobic ponds were not operated), the BOD removal was reported to be 85 percent. The same author also reported that in another multi-cell anaerobic pond in Sutherland in Saskatchewan, the overall BOD reductions in winter were 24 percent compared with summer removals of 43 percent. Typical variations in BOD in cold climate lagoons over a year as indicated by Fisher et al (1) is presented in Figure 8. According to these authors, the curve indicates a slow build-up of BOD in the pond concurrent with the formation of ice and subsequent snow cover and a rapid reduction in BOD during the spring recovery period.

The BOD reductions in the single-cell anaerobic lagoons were lowest with 48.8 to 52.7 percent as against 88.8 to 73.1 percent achieved in a two-cell facultative lagoon. Thus it is clear that single-cell anaerobic lagoon performance is comparable only to that of a primary treatment unit. According to Fisher et al (1), pond efficiencies, based on BOD reductions in Alberta varied between 45 and 90 percent, and in Saskatchewan summer BOD reductions averaged in excess of 90 percent while winter efficiencies averaged 54 percent.

FIGURE 8
Typical Variations in BOD in Cold Climate Lagoons (1)



SS Removal

The suspended solids reductions (based on typical values for the raw wastewater) in the multi-cell facultative lagoon were high for fall discharge (82.5 percent) compared to spring discharge (70 percent). Among the three types of lagoon systems studied, the single-cell anaerobic lagoons provided the least treatment in reducing SS levels (48.5 percent).

The removal of SS from wastewater stabilization ponds in Saskatchewan reported by Fisher et al (1) was 70 percent during summer operation and 77 percent during the winter. While the summer time BOD and SS reductions were comparable to a secondary treatment facility, the treatment efficiencies reduced significantly during the winter period due to ice cover.

Reductions of Nutrients

In the case of P, highest reductions of 85 and 50 percent occurred in the multi-cell facultative lagoon during fall and spring discharges respectively and the lower percentage level of 34 was achieved in the single-cell anaerobic lagoon. Highest reduction of 50 percent was achieved in the case of two-cell facultative lagoons.

In the case of TKN, the highest reduction of 68.5 percent was observed in the multi-cell facultative lagoon discharging in fall compared to 37.5 percent achieved in the multi-cell facultative lagoons discharging in spring. This indicates that TKN reductions were good during summer time rather than winter time hold-ups. Lowest reduction in TKN was achieved in single-cell anaerobic lagoons (11.3%) compared to two-cell facultative lagoons performance (43.9 %).

Coliform Levels

The results from various lagoon systems indicated that total coliform levels were < 5.5×10^4/100 mL in the fall discharges of multi-cell facultative lagoons compared to the maximum level of > 2.5×10^6/100 mL in the effluent from single-cell anaerobic lagoons. Clark et al [22] indicated that the coliform removal efficiencies during winter in northern regions ranged from 97 to 99 percent.

CONCLUSIONS

The following general conclusions can be drawn based on the study:

1. Of the three types of lagoon systems studied viz., single-cell anaerobic, two-cell facultative and multi-cell facultative, multi-cell facultative lagoons performance was comparable to secondary treatment units in achieving BOD and SS reductions.

2. The single-cell anaerobic lagoons performed more like primary treatment units.

3. In the case of multi-cell facultative lagoons discharging twice a year, the fall discharges were of better quality than the spring discharges.

4. Maximum reductions in TKN and P occurred in the multi-cell facultative systems discharging in fall, 66.5 and 65 percent respectively.

5. Minimum coliform concentrations were observed in the multi-cell facultative lagoon effluent discharging in fall (< 5.5×10^4 per 100 mL).

6. Detailed study of the data for all the pollutant parameters, under different discharge conditions and periods will have to be undertaken for a better understanding of the performance of lagoons in Saskatchewan, and to develop strategies for lagoon effluent upgrading that may be required in the next decade.

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ONE YEAR IN THE LIFE OF A VERY POOR WATER SUPPLY:
An Investigation of Odour Problems in Driedmeat Lake, Alberta

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INTRODUCTION

Many prairie communities obtain their water from small lakes or sloughs which are filled primarily by surface runoff. These reservoirs are often nutrient-rich and support a prolific growth of plants and animals. In addition to nutrients, the water may contain significant concentrations of sodium, sulphate and bicarbonate which impart their own tastes to the water. In many treatment installations heavy chlorination is required to maintain adequate chlorine residuals in the presence of organic compounds producing a 'chemical' taste.

During the winter these reservoirs usually become anaerobic which allows iron and manganese to be released from the sediments and the production of hydrogen sulphide resulting from the bacterial reduction of sulphate.

These factors produce an aesthetically poor water which requires either tolerance on the part of the consumer or advanced treatment techniques to produce an acceptable product.

The City of Camrose Alberta obtains its water from Driedmeat Lake located 13 km south of the city. The water taken from the lake is of reasonable quality:

Sodium	80 - 90 ppm
Sulphate	100 - 200 ppm
$NO_3 + NO_2$	0.80 ppm
T.D.S.	300 ppm

The original system passed the lake water through a gravel aquifer followed by chlorination. As the population increased the capacity of the aquifer became inadequate to supply the city. High manganese concentrations resulted in citizens complaining about laundry stains.

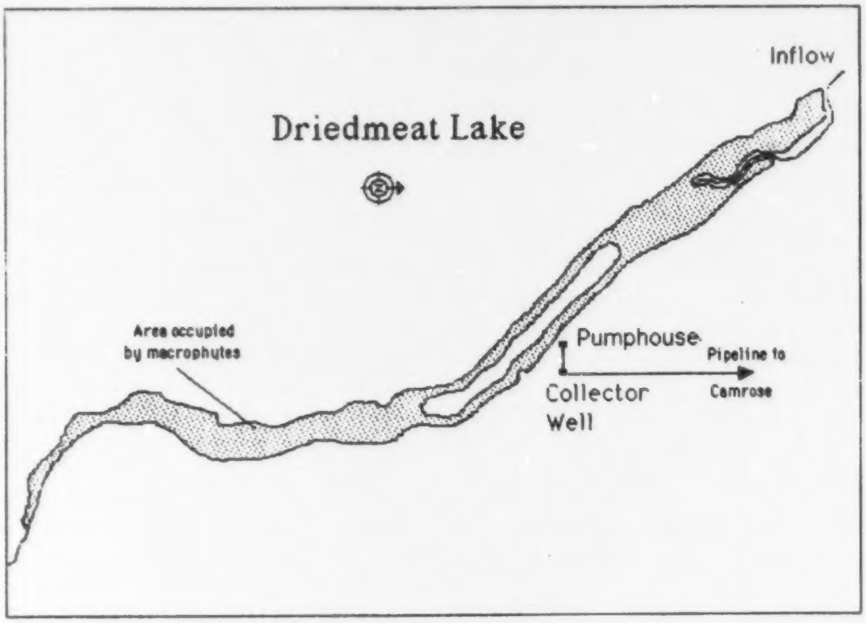
In December 1987 a new treatment plant was commissioned which uses alum flocculation in two clarifiers with softening and pH adjustment.

The first complaints about bad taste and odours from water treated in the new installation were received in July of 1988. This incident was of short duration and may have been caused by stagnant water in one of the mains. The treated water was of acceptable quality until October 21, 1988 when numerous complaints were received. The odour was particularly noticeable in showers and was described as musty or earthy.

Driedmeat Lake is an enlargement on the Battle River with an area of 906.5 ha. It has a mean depth of 1.8 m with a maximum depth of 3.6 m and a volume of 16.4 million cubic metres (Figure 1).

Submersed macrophytes (rooted aquatic plants) occupy 805 ha or 87% of the lake area, reaching the surface making boating impossible in many areas of the lake. An algal bloom occurs throughout the summer.

FIGURE 1
Location of water intake, pumphouse and collector well



A biological study was started in August 1988 to examine some of the biological communities to see how they might affect the quality of the water. The lake was visited weekly through the summer and autumn and then biweekly over the winter. Water samples obtained were collected from three areas: 1. the lake, 2. after the micro-screens, and 3. from the raw water at the treatment plant.

Chlorophyll a was measured by extraction with dimethyl formamide following filtration. Algae was identified and enumerated by counting using an inverted microscope. Oxygen concentrations were measured using an oxygen electrode.

The macrophyte community was observed from a boat until freeze-up. No quantitative measurements were made.

Water samples collected during the winter when taste and odour was present were used to determine which techniques could be used to remove these compounds. These included additions of potassium permanganate, ozone, activated carbon and aeration.

RESULTS AND DISCUSSION

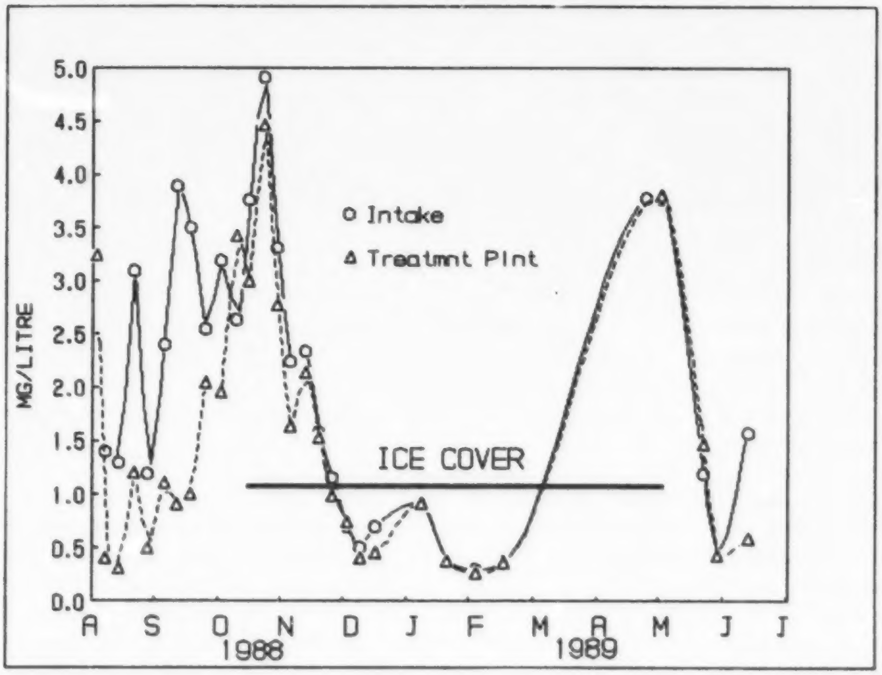
Phytoplankton Growth

Chlorophyll a content of the water at the lake intake and in the raw water at the treatment plant is shown in Figure 2. The differences in concentration seen between the two sites in August through October are due to the removal of Cyanobacteria by the micro-screens at the lake pumphouse. These species form large colonies which are retained by the 35 micron screens.

Sixty to eighty percent was retained. As single-celled species became more prevalent in November, very little algae was removed. Passage through the screens and pumps caused some cell disruption and an increase in odour was found in water samples collected after the screens. Efficient removal of algal cells would prevent them from decomposing or from leaking compounds into the water during their passage through the pipeline to the treatment plant.

FIGURE 2

Chlorophyll a content of water at the lake intake and treatment plant raw water



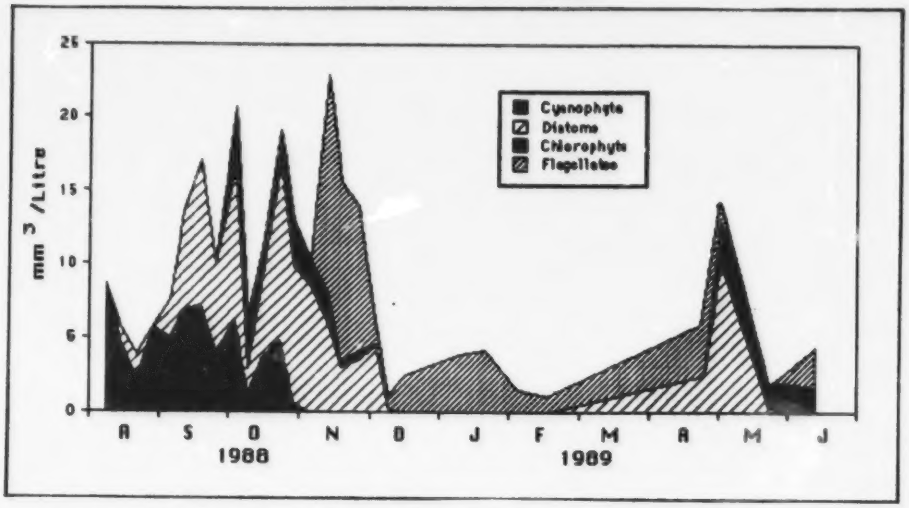
Driedmeat Lake supports a heavy growth of algae throughout the summer consisting primarily of Cyanobacteria (blue-green algae). The dominant species found were Aphanizomenon flos-aquae, Microcystis aeruginosa, and Anabaena spiroides. These Cyanobacteria are usually associated with taste and odour but during August, September and October when they were abundant, taste and odour was not a problem in the treated water. Some odour was present in the raw water but was removed by the treatment plant. The algal bloom as indicated by cell volumes of four different algal groups as given in Figure 3 was greatest in late October just before the ice formed. At this time the diatoms were rapidly replacing the Cyanobacteria in abundance. These consisted largely of Stephanodiscus, Cyclotella with lesser amounts of Nitzchia.

During the winter the diatom Cyclotella and the flagellated golden-brown alga (Pyrrophyta) Cryptomonas became dominant.

As soon as the ice went out in early May a short-lived bloom of flagellates (Chlamydomonas and Rhodomonas) appeared. The abundant nutrients, rising temperatures and light encouraged this growth. Large numbers of Daphnia appeared and their predation most likely contributed to the rapid decline in the spring bloom.

FIGURE 3

Species composition at pumphouse intake
expressed as cell volumes



By early June 1989 the Cyanobacteria were beginning to grow rapidly and would be expected to become dominant within a couple of weeks.

The occurrence of taste and odour could not be related to the abundance of algae as indicated by chlorophyll concentration or to the presence of any particular species.

The shallowness of the lake combined with its alignment with the prevailing winds causes complete mixing of the water. Temperature is uniform from top to bottom most of the time and the temperature can change rapidly with changes in air temperature (Figure 4). In October 1988 the lake temperature fell from 10^0 to 5^0 in less than two weeks. By the time ice formed, the water had cooled to 2^0 and reached a recorded minimum of 0.7^0 in late February. In deeper lakes the bulk of the water remains at 4^0 through the winter. The temperature rose rapidly in early May when the ice broke up and was soon high enough to enable the macrophytes to begin growth.

Oxygen concentration of the water from the lake to the treatment plant is illustrated in Figure 5. Concentrations approached saturation during the ice-free part of the year but fell gradually under ice cover. By early February there was no detectable oxygen. The smell of hydrogen sulphide was quite strong in the pumphouse beginning in early February. Some aeration occurs as the water passes through the micro-screens and at the collector well where it falls through several metres entraining air in the process. Because of the low temperature sufficient oxygen was taken up to destroy the sulphide and bring the oxygen concentration to near saturation.

As the water travels through the pipeline to the treatment plant oxygen concentration decreases. The decomposition of micro-organisms and other oxidative processes lower the concentration by about 1 to 2 mg/litre. The oxidation of the sulphide consumed about 5 mg/litre in February.

The submersed aquatic plants form the largest community (in terms of biomass) in Driedmeat Lake. Eleven species have been found and these occupy 2/3 of the lake area. The most common species are *Potamogeton richardsonii*, *P. vaginatus*, *Myriophyllum exalbescens* and *Ceratophyllum demersum*. These plants grow to a depth of 2 to 2.5 metres and frequently reach the surface.

FIGURE 4

Lake temperature at pumphouse intake and treatment plant

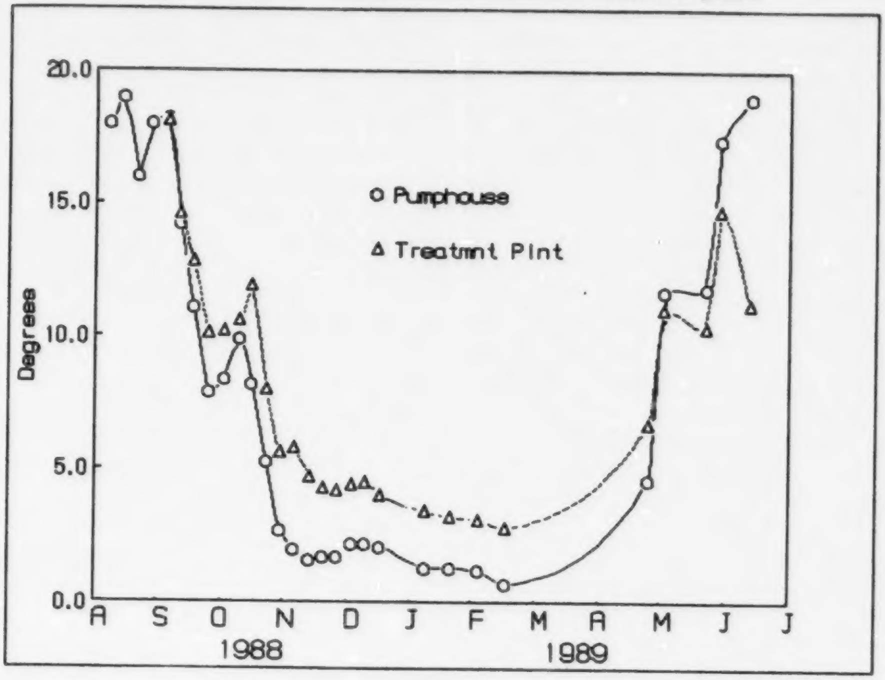
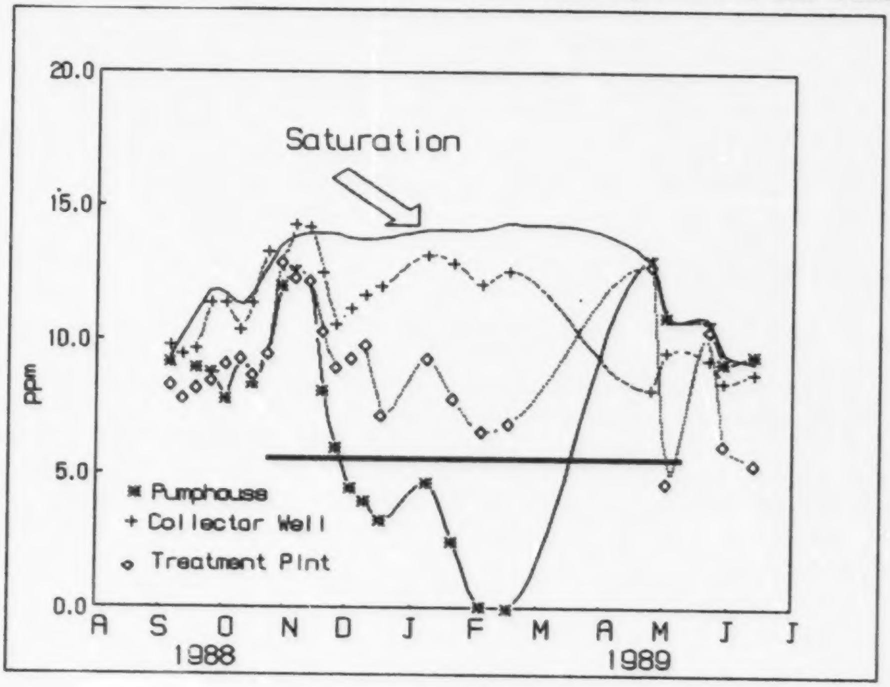


FIGURE 5

Oxygen concentration at the pumphouse intake,
collector well and the treatment plant.
The saturation concentration at the current intake temperature is also shown.



All of these species overwinter as dormant shoots which lie on the bottom. Active growth begins when the water temperature rises shortly after the ice break up usually in late April. Floating shoots of Myriophyllum with adventitious roots were seen in early May. Initial growth is very rapid and slows when the plants reach the surface. Detached plants are often moved by the wind and deposited on the shore near the intake of the water system.

By mid-October, the plants were beginning to turn brown and more were detaching or fragmenting. During the winter the plants decompose leaving the overwintering structures. The decomposition of the large quantity of macrophytes in the lake after freeze-up reduces oxygen concentration and releases organic compounds into the water. Also the organisms responsible for the decomposition can produce compounds which can impart taste and odour to the water.

Removal of Taste and Odour

Several simple qualitative experiments were carried out to determine how the taste and odour present during the early winter could be removed.

The earthy odour could be completely removed by purging with air. At 14 degrees, 200 volumes of air per volume of water bubbled through a column 38 cm high was sufficient to remove the odour. At 2° the odour was not removed with 12 hours of aeration.

During the summer and autumn, aeration in the collector well and at the treatment plant probably reduce the concentration of odoriferous compounds significantly. As the water and air temperatures fall this mechanism becomes ineffective. This would occur at the same time that the macrophytes are decaying.

The addition of at least 20 ppm of activated carbon was effective but not consistent particularly at 4° or less, but its action is not affected by temperature as much as aeration.

It was difficult to achieve a high enough carbon load in the treatment plant clarifiers to adequately remove odours in the winter of 1988-89. The floc tended to float to the top where it was lost. The use of polyelectrolytes has reduced this problem somewhat.

Treatment of the water with ozone at 15° and 2° did not reduce the odour.

Potassium permanganate was added at a rate of up to 5 mg/litre. It was decolourized but did not reduce the odour. Permanganate was added for a time at the pumphouse but did not have a noticeable effect.

Origins of Taste and Odour

The occurrence of taste and odour in July of 1988 was of short duration and its cause was not determined. For the remainder of the summer no complaints were received even though a fishy, cut grass smell was present in the collector well and the aeration chambers of the treatment plant. The treatment process effectively removed these odours.

Taste and odour did not appear to be related to the type or quantity of algae present in the lake but appeared to be connected with the decomposition of the macrophytes. A simple experiment was carried out to determine whether the sediments or decaying macrophytes was the origin of the taste and odour.

A water sample containing algae and having an earthy smell was aerated until the smell disappeared. The water was incubated in the light at about 12-15° for one week then checked for the return of the odour. The taste and odour did not return and it was concluded that the algae was not the prime source.

A sample and sediment and plant material was collected on December 19 from beneath the ice using an Eckmann dredge. The lake water was aerated at room temperature for several hours to remove volatile materials. One litre of this water was placed in a jar along with enough sediment to form a layer about 2 cm thick. A similar container was set up containing plant material. This consisted of old stems of Myriophyllum and winter buds which were still attached to the stems. These, along with a third container containing only water were kept at 2° in the dark for two weeks. An air space was present in each jar to maintain aerobic conditions. At the end of the incubation, the water was warmed, smelled and tasted. The earthy smell was present at a low level in the water in contact with the plant material but was not detected in the water from the other two treatments.

This experiment would indicate the likelihood that decaying macrophytes were the origin of the musty odour. The compounds responsible are probably produced by the organisms carrying out the decaying processes.

Summary

Driedmeat Lake provides water which is acceptable on a chemical basis but is aesthetically unacceptable at certain times of the year because of bad taste and odour. The onset of these incidents has been sudden and so far have not been predictable.

Taste and odours of biological origin have been extensively investigated concentrating on two compounds: geosmin and 2-methylisoborneol. These compounds are produced by some strains of Cyanobacteria (Slater and Blok, 1983) and by Actinomycetes (Gerber, 1983 and Yagi et al, 1981). Juttner and Wurster (1979) detected up to 80 different compounds released by algae. Some strains of Cyanobacteria produce toxins which are largely retained inside the cells and are thus removed by filtration in the treatment process. These toxins can be of concern when heavy blue-green blooms regularly occur. Chronic liver damage was correlated with algal bloom in Malpas Dam reservoir supplying Armidale N.S.W. Australia (Falconer et al, 1983).

The removal of taste and odours in the treatment plant is difficult. Aeration is effective when the water is warm enough but in the winter, little is removed. Chemical oxidation by chlorine, ozone or permanganate was not effective. Only activated carbon, if present in adequate amounts, brought about a reduction.

Driedmeat Lake could be improved both as a water source and for fishing and boating by removing some of the macrophytes during the summer. This would reduce the amount of biomass that decays during the fall and winter. Macrophyte harvesting can remove nutrients from a lake and may gradually improve its quality. Harvesting has been carried out in the eastern end of Lake Wabamun, located west of Edmonton since 1972. In 1987 the cost was $260/ha harvested when 275 tons were removed (Transalta Utilities).

Water of acceptable quality from sources like Driedmeat Lake is possible when good treatment facilities combined with skilled operation is used. In spite of this water of unacceptable quality can be released when the treatment facilities are overwhelmed or cannot respond quickly enough. A source of good quality water is still the most effective way of maintaining a good quality product.

ACKNOWLEDGEMENTS

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INDICATOR BACTERIA IN CALGARY TAP WATER

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INTRODUCTION

The City of Calgary operates two water treatment plants, both of more than 700 ML/d capacity. The Bearspaw Water Treatment Plant draws source water from the Bow River, while the Glenmore Water Treatment Plant purifies Elbow River water from the Glenmore Reservoir impoundment. While finished water from either plant can be distributed virtually to the entire city, on an average demand day, Glenmore Water Treatment Plant provides water primarily to the southern parts of the City while Bearspaw Water Treatment Plant serves the northern sections.

Compliance and quality assurance monitoring of Calgary's finished and distribution system water consist of collection and analysis of water samples from pre-selected sites at scheduled times. The distribution monitoring program is designed to sample water representative of the transmission and distribution mains (1,2). Taps are flushed prior to sample collection to ensure that water being collected originates from outside the property line of the sampling site.

With this extensive distribution system monitoring program in place, one might assume that the bacteriological quality in Calgary's distribution system is so well characterized that any unusual deteriorations in quality could be easily recognized and remedied. The distribution system program, however, is designed to monitor water quality in the large distribution mains. Although the routine monitoring network geographically covers the City and its population well, sampling sites were intentionally chosen which receive water from large distribution mains rather than the smaller distribution lines which bring water to private homes. The distribution system is so large in a city like Calgary with over 650,000 people, that it becomes impossible to routinely monitor water quality in the fine network of distribution system pipes which service individual homes or small groups of homes. Thus, Calgary's distribution monitoring program provides extensive information on distribution system water quality only on a "macro" scale rather than a "micro" scale.

Private homes are sampled only in response to complaints. If the samples are positive for total coliforms (TC), follow up samples are collected from the same home as well as from homes and hydrants in the immediate vicinity. Typically, the percent of TC and E. coli recoveries from these complaint samples exceeds that obtained in the "macro" scale monitoring program. Since little is known about typical water quality in the small feeder mains which bring water to these homes, it is very difficult to evaluate these results. Are these higher coliform isolation frequencies in home samples indicative of a major problem in the distribution system in the area or are the results typical for small mains?

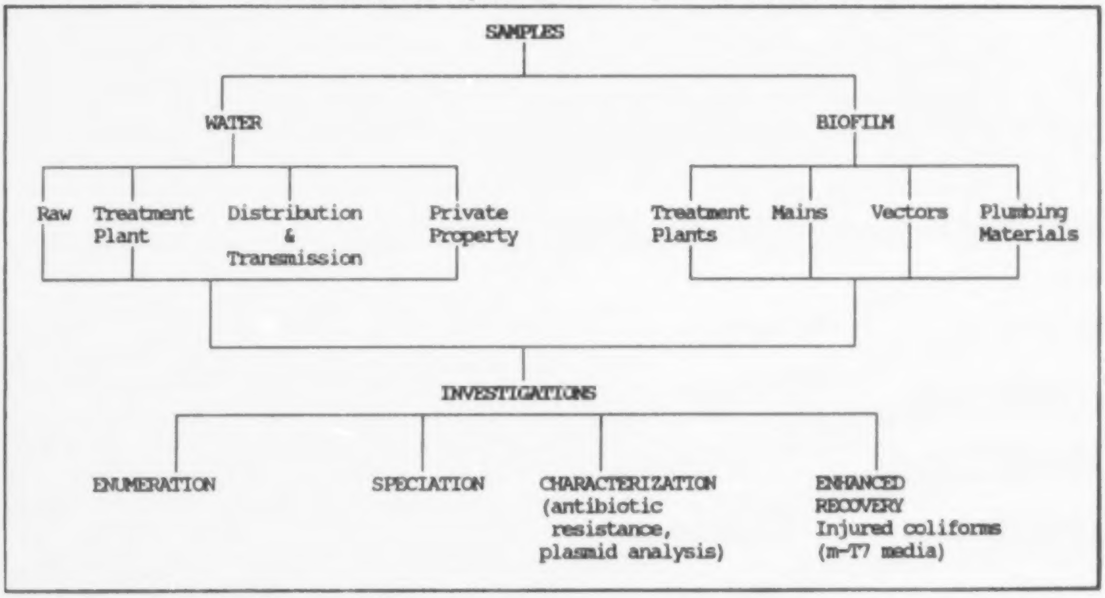
During January, February and March, 1987 coliform bacteria were detected in homes in one isolated district in Calgary during sampling initiated in response to a taste and odour complaint (3). Both TC and E. coli were recovered in taps in several private homes. Such isolations were unprecedented in historical distribution system sampling records. Meanwhile, daily microbiological monitoring tests in the city wide network did not alert laboratory staff to the isolated problem developing in the area. The lack of a clear explanation for occurrences of coliforms indicated a need for research into the causes of the problem.

STUDY OBJECTIVES

Many water utilities have from time to time been confronted with positive coliform tests that defy explanation. If coliform incidents in Calgary from December 1986 to March 1987 (3) resulted from intrusion of contamination or ineffective treatment, the cause must be addressed. If a more thorough study of the distribution system indicated that TC and E. coli in low numbers are ubiquitous, this situation would be undesirable but less urgent. If contamination originated from the water itself, the source of bacteria should be pinpointed. If it originated from biofilm, causes of sloughing should be defined and avoided.

In order to prevent future incidents of poor bacteriological water quality in the distribution system, a thorough investigation was launched. An in-depth survey of bacterial populations was planned to supplement the routine microbiological monitoring program. As shown in Figure 1, samples were to be collected from more environments, more frequently and in larger volume. A broad spectrum of specific tests were being conducted on these intensive samplings to better characterize the source and nature of coliform organisms in water throughout the treatment plants and in the city distribution network.

FIGURE 1
Sampling strategy and analytical program
for distribution system microbiological evaluation


SAMPLES
WATER
BIOFILM
Raw
Treatment Plant
Distribution & Transmission
Private Property
Treatment Plants
Mains
Vectors
Plumbing Materials
INVESTIGATIONS
ENUMERATION
SPECIATION
CHARACTERIZATION (antibiotic resistance, plasmid analysis)
ENHANCED RECOVERY Injured coliforms (m-T7 media)

The study's objectives were, firstly, to compare water quality results from four different monitoring programs: 1) Macro-scale distribution system monitoring, 2) Micro-scale distribution system monitoring, 3) Water main repair monitoring and 4) Monitoring during an incident of poor bacteriological quality in the distribution system. Secondly, the sources and nature of indicator bacterial populations in Calgary's treated water supply were to be identified.

Samples were collected for analysis of heterotrophic organisms by Standard Plate Count (4) and TC to determine whether these two parameters: 1) varied seasonally, 2) varied between older and newer areas of the City, 3) were affected by sampling intensity, 4) were affected by contaminated plumbing fixtures, and 5) differed in areas receiving water from Bearspaw Treatment Plant or Glenmore Treatment Plant.

This study is presently still underway. This paper describes preliminary results, with emphasis on the frequency of TC isolation and the species identification of these TC organisms.

SAMPLE COLLECTION STRATEGY

Macro-scale distribution system monitoring.

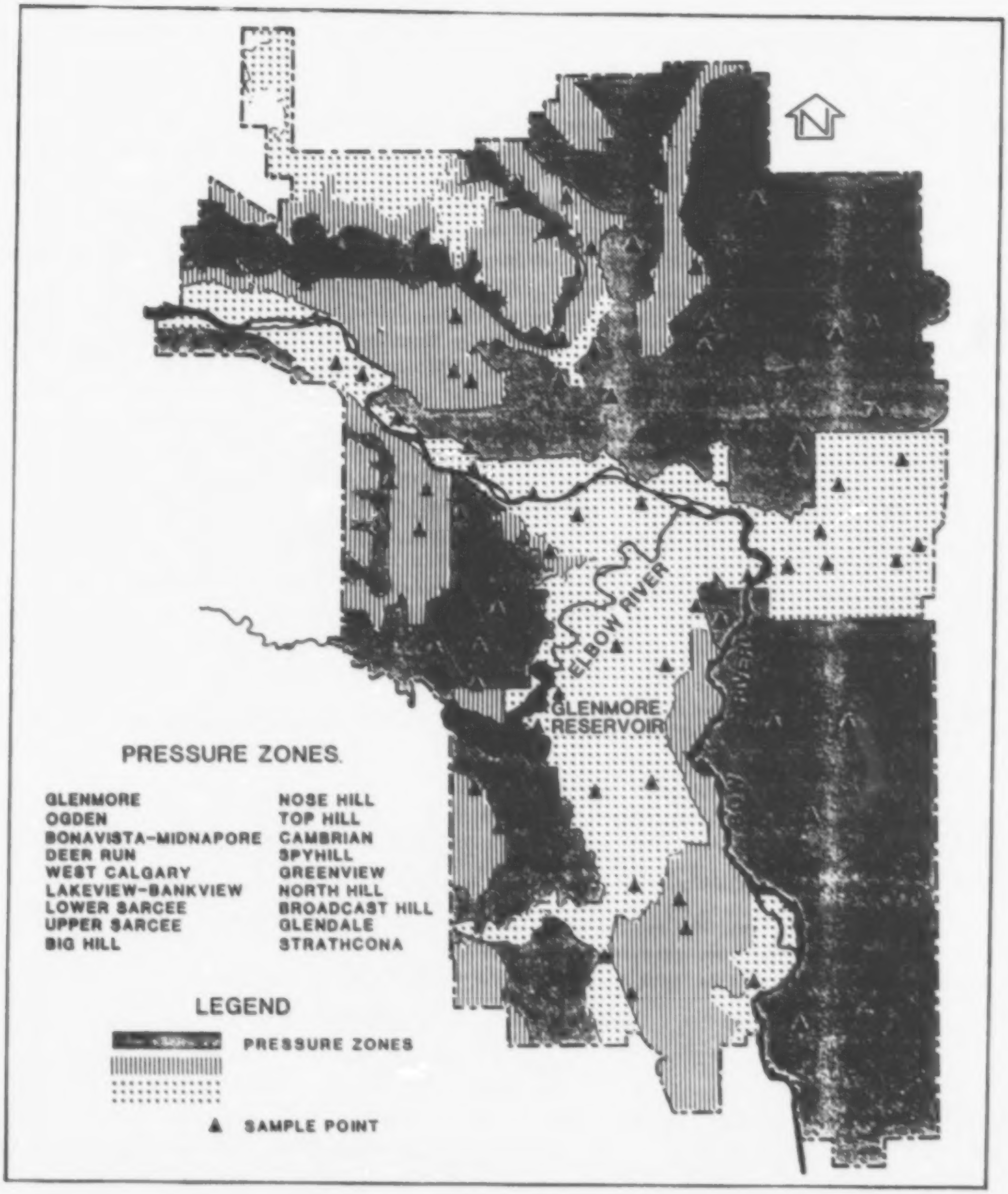
The City of Calgary has a network of sampling sites located throughout the city which have been selected to identify any deterioration in water quality after treatment. The network was originally designed to fulfill microbiological monitoring requirements recommended by Health and Welfare Canada (5). Every week 59 samples are collected from distribution sites (Figure 2) and analyzed at the Provincial Laboratory of Public Health (Southern Branch, Calgary) for Heterotrophic Plate Count, TC and fecal coliforms. This sampling frequency exceeds Provincial guidelines (6) requiring that 36 samples be collected weekly from a city the size of Calgary. Locations include all pressure zones and the number of sites per pressure zone is proportional to the number of residents in the zone. Forty-seven of the 59 samples are collected from fixed locations while the remaining 12 samples are obtained from 24 alternating sites. From December 15, 1988 to April 30, 1989, 988 such samples were collected for bacteriological monitoring. From ten of these sites located nearest the extremities of the distribution system, free and total chlorine measurements were made using amperometric titration. From ten different sites, simple chemical and physical tests (including temperature, pH, conductivity, turbidity, alkalinity, and hardness) were conducted monthly, trihalomethanes (chloroform, bromoform, bromodichloromethane, chlorodibromomethane) were monitored biweekly, and metals (aluminum, copper, lead, cadmium chromium, iron, nickel) were analyzed quarterly.

Sampling sites were selected to meet three criteria: 1) to make the percentage of samples collected per pressure zone equal to the percentage of the total city population which resides in each zone, 2) to provide even geographical coverage throughout the city, and 3) to facilitate the collection of water from the transmission and distribution mains. Samples were collected from public and commercial buildings, from heavy users of water whenever possible. Sampling sites include fire stations, libraries, athletic facilities, and hospitals.

Samples were collected from the same tap location every week. If the tap had an aerator or screen on it, these attachments were removed. The tap was flushed at a high flow rate for three minutes, and then at a low flow rate for an additional two minutes prior to sample collection.

FIGURE 2

City of Calgary distribution system
water quality monitoring network covers all
pressure zones and is proportional to area population


ELBOW RIVER
GLENMORE RESERVOIR
BOW RIVER
N
PRESSURE ZONES.
GLENMORE
OGDEN
BONAVISTA-MIDNAPORE
DEER RUN
WEST CALGARY
LAKEVIEW-BANKVIEW
LOWER SARCEE
UPPER SARCEE
BIG HILL
NOSE HILL
TOP HILL
CAMBRIAN
SPYHILL
GREENVIEW
NORTH HILL
BROADCAST HILL
GLENDALE
STRATHCONA
LEGEND
PRESSURE ZONES
SAMPLE POINT

Micro-scale distribution system monitoring.

Micro-scale monitoring was designed to investigate the occurrence of coliform organisms when samples were collected from many different sites located in close proximity to each other. This objective was achieved by sampling private residences. Four areas of the city were selected for micro-scale monitoring. Two areas, one older district (G-OLD) and one newer (G-NEW) district, were located equidistant from Glenmore Water Treatment Plant. Two other areas, one older district (B-OLD) and one newer district (B-NEW), were located equidistant from Bearspaw Water Treatment Plant.

On three or four days each week, samples for microbiological analysis were collected from two of the four districts. In each district, two streets were randomly selected and samples were collected from ten residences on each street. In addition, "control" samples were collected from three locations in each of the two districts. Thus, 26 sites were sampled each day. The "control" sites were located in public buildings similar to those from which macro-scale distribution system samples were collected. The "control" samples were collected from the same location every day. From December 15, 1988 to April 30, 1989, 1,109 samples had been collected for microbiological analysis. A total of 216 of the 1,109 samples were collected at the control sites and the remaining 893 were collected from homes in the four neighborhoods. Chlorine titrations were performed at two of every five sampling sites. Samples were collected for chemical and physical analysis from one of every five sites. Samples were collected with screen/aerator removed (whenever possible) after a five minute flush at high flow rate.

Water Main Repair Monitoring.

All water mains which had undergone depressurization for replacement or repair were monitored for microbiological water quality after the work had been finished and flushing carried out. These samples were typically (95% of the time) collected from hydrants and involved no building plumbing. From May 1, 1988 to December 31, 1988, 950 samples were collected in this monitoring program.

Distribution Monitoring during a Coliform Outbreak.

An unusual number of TC were recovered from a small geographic area in the City of Calgary in the first few months of 1987. For purposes of comparison with the other data sets described in this paper, data gathered during this outbreak were compiled. The data set differs from those previously described in two ways. Firstly, not all sheen colonies from the membrane filtration test were speciated. Thus, when species information is discussed it is based only on the subset of samples which were subjected to speciation. Secondly, early in the coliform outbreak samples were collected from taps from which screens, aerators, and point of use devices had not been removed. Data which is presented here does not include results of samples collected from taps which had aerators and point of use filters connected. However, it may include results of samples collected from taps which had screens on, as it was not possible to differentiate between these two types of samples. A couple of weeks into the outbreak, when it was realized that the screens may have been contributing to erroneous results, no samples were collected from taps with screens on.

Resampling of Total Coliform Positive Sites

If less than 10 sheen colonies per 100 mL or more than 5 non-sheen colonies were detected on the mEndo agar, a second sample was collected from that location. If more than 10 sheen colonies per 100 mL were present on the mEndo agar, the distribution system pipes in the area were flushed by releasing water from the fire hydrant for 15 to 25 min. The original site was then resampled and samples were also collected form several homes in the vicinity as well as from the hydrant.

SAMPLE ANALYSIS

Samples were collected with sterile 200 mL plastic bottles with sodium thiosulfate added as a chlorine neutralizer (Systems Plus, New Hamburg, Ontario). Samples were promptly placed into an iced cooler, transported to the laboratory, and analyzed within 4 h of collection.

Total coliforms were analyzed (4) by single step direct membrane filtration technique (*Standard Methods* Section 909A) using mEndo Agar LES (Difco, Detroit, MI) and 0.45 um membrane filters (Gelman, Ann Arbor, MI). Species identifications were performed on sheen colony organisms using the API-20E kit (Analytab Products Ltd., St. Laurent, Quebec) after culture purification on Tryptic Soy Agar (Difco).

All analyses were conducted according to *Standard Methods* (*SM*) (4). Chlorine measurements were made by amperometric titration (*SM* Method 408C) using a Wallace and Tiernan titrator. pH was measured using a Fisher Accumet 750 Selective Ion Analyzer (*SM* Method 423), and specific conductance was measured (*SM* Method 205) using a Beckman Conductivity Bridge. Hardness and alkalinity titrations were performed as described in *SM* Methods 314B and 403, respectively. Nephelometric turbidity was measured using *SM* Method 214A with a Sigrist photometer.

RESULTS

Macro-scale monitoring.

Of the 988 samples collected and analyzed for the macro-scale monitoring program (December 15, 1988 to April 30, 1989) only two (0.20%) were positive for total coliforms (Table 1). Neither of the two samples contained *E. coli*. Subsequent re-samplings of the two TC positive sites were TC-negative. Chlorine residuals measured during this time period ranged from 0.16 mg/L to 0.38 mg/L, with a mean of 0.26 mg/L.

Micro-scale monitoring.

Between December 15, 1988 and April 30, 1989, 1,109 samples were collected and analyzed for the micro-scale monitoring project (Table 1). A total of 216 of these 1,109 samples were collected from four control sites, one in each of the four neighborhoods (G-NEW, G-OLD, B-NEW and B-OLD). Chlorine residual in the control samples ranged from 0.12 mg/L to 0.42 mg/L with a mean of 0.28 mg/L. Total coliforms were never recovered from any of these 216 control site samples. The remaining 893 samples were collected from homes in the four neighborhoods. Chlorine residual in the home samples ranged from 0.11 mg/L to 0.41 mg/L with a mean of 0.30 mg/L. Fifteen, or 1.7%, of these home samples were positive for total coliforms. Three of fifteen samples (0.3% of the 893 samples and 20% of the 15 TC-positive samples) were positive for *E. coli*. TC counts in these samples ranged from 1 Colony Forming Unit (CFU)/100 mL to 150 CFU/100 mL, with a mean of 14 CFU/100 mL (Table 2); *E. coli* enumerations ranged from 2 to 140 CFU/100 mL. None of the resamples were TC-positive or *E. coli*-positive.

TABLE 1

Incidence of Total Coliform isolations, number of different Total Coliform species identified and number of *E. coli* isolated from distribution system water samples.

RESULTS	DISTRIBUTION SYSTEM SAMPLING PROGRAM				
	Macro scale Monitoring	Micro Scale Monitoring		Water Main Repair Monitoring	Coliform Outbreak
		Control	Home		
Total Number of Samples	968	216	893	952	2,828
Total Coliform Isolations (% of Total Sample numbers)	0.20%	0%	1.7%	2.2%	3.1%
E. coli Isolations (% of Total Sample numbers)	0%	0%	0.3%	0.2%	1.2%
E. coli Isolations (% of Total Coliform positive samples)	0%	0%	20%	9.5%	39%
TC Isolations from resamples[1]	0%	0%	0%	4.8%	39%

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[1] Sites positive for Total Coliforms (TC) were resampled. The number is this column represents the numbers of resamples which were also TC positive, expressed as a % of the total number of resamples.

TABLE 2

Number of Total Coliforms (CFU/100 mL) and number of *E. coli* (CFU/100 mL) isolated from four distribution monitoring programs

MONITORING PROGRAM	TOTAL COLIFORMS (CFU/100 mL)				# of Species Recov-ered	*E. coli* (CFU/100 mL)			
	No. of Positive Samples	Minimum	Maximum	Mean		No. of Positive Samples	Minimum	Maximum	Mean
Macro-scale	2	2	>200	–[1]	n.s.[2]	0	0	0	0
Micro-scale	15	1	150	14.0	10	3	2	140	48
Water Main	21	1	45	5.1	14	2	1	11	6
Coliform Outbreak	25	1	>200	23.0	4	9	1	>200	88

The organism recovered from the greatest number of sites (seven) was *Enterobacter cloacae*. *Klebsiella pneumoniae* and *E. coli* were each recovered from three sites. Other species information is presented in Table 3. Of the eleven sites from which more than 1 TC/100 mL were recovered, six sites had more than one species of TC identified and five sites had only one species of TC identified.

TABLE 3

Number and type of species isolated from 15, 21, and 25
Total Coliform positive water samples collected, respectively,
from micro-scale, water main repair and coliform outbreak monitoring.
("-" denotes none detected)

COLIFORMS (GENUS AND SPECIES)	SPECIES ISOLATED FROM DISTRIBUTION SYSTEM MONITORING PROGRAMS		
	Micro-Scale Monitoring	Water Main Repair	Coliform Outbreak
Number of different Species Identified	10	14	4
CITROBACTER			
C. amaloniticus	7	-	-
C. freundii	-	10	-
ENTEROBACTER			
E. aerogenes	-	5	-
E. agglomerans	13	38	-
E. amnigenus	-	10	-
E. cloacae	46	-	28
E. intermedium	7	5	-
E. sakazakii	7	-	-
E. species	-	5	-
ESCHERICHIA			
E. adecarboxylata	13	5	-
E. coli	20	10	36
E. hermanii	-	5	-
E. vulneris	-	5	-
KLEBSIELLA			
K. oxytoca	13	5	-
K. pneumoniae	20	-	32
KLUYVER			
K. cryocrescens	-	5	-
SERRATIA			
S. liquefaciens	-	10	-
S. odorifera	7	-	-
S. species	-	5	-
CDC GROUP 17	-	-	3

Water main repair monitoring.

Results from water main repair monitoring were taken from samples collected between May 1, 1988 and December 31, 1988. This is because these samples were analyzed using the membrane filtration method, yielding data directly comparable with the macro- and micro-scale monitoring data and with the coliform outbreak data. After December 31, 1988, the method for routine analysis of TC in water main monitoring samples was changed to then Autoanalysis Colilert test in the presence absence format (7).

Between May 1, 1988 and December 31, 1988, 950 samples were collected and analyzed. Of these samples, samples from 21 sites (or 2.2%) were positive for TC (Table 1). The 21 samples included two samples (0.2% of the 950 and 9.5% of 21) which were positive for *E. coli*. None of the resamples from TC-positive sites were positive for TC or for *E. coli*. Total coliforms enumerated in these samples ranged from 1 CFU/100 mL to 45 CFU/100 mL with a mean of 5.1 CFU/100 mL (Table 2); *E. coli* enumerations ranged from 1 CFU/100 mL to 11 CFU/100 mL. The organism recovered from the greatest number of sites was *Enterobacter agglomerans* (Table 3).

Coliform outbreak.

Between January 18 and June 18, 1987, 2,828 samples were collected and analyzed (Table 1) which related to the coliform outbreak and to follow-up resampling. A free chlorine residual was always present in samples collected from homes in the affected neighborhood. Of the 2,828 samples, 88, or 3.1%, were positive for TC. *E. coli* were recovered from 34 of the 88 samples (or 1.2% of the 2,828, 39% of the 88 TC-positive samples). The 88 samples were collected from 51 sites; 20 of the 51 sites were TC positive upon resampling. As summarized in Table 2, TC counts as well as *E. coli* counts ranged from 1 to over 200 CFU/100 mL in water samples collected from homes during the coliform outbreak.

Not all TC in this group of organisms were speciated. TC from 28 of the 88 TC-positive samples were identified (Table 3). *E. coli*, the organism most frequently recovered, was isolated from 11 (39%) of the 28 speciated samples. *Klebsiella pneumoniae* and *Enterobacter cloacae* were recovered from 8 (29%) and 7 (25%) of the 28 speciated samples, respectively. Further information on species recoveries is shown in Table 3. Looking at the results of the resamples only, *K. pneumoniae* and *E. cloacae* made up 5 (38%) and 4 (31%), respectively, of the 13 resamples whose TC were speciated. In 18 of 18 (100%) samples which recovered >1 TC/100 mL and which were speciated, all of the organisms recovered on the membrane filtration plate were the same species.

DISCUSSION

Many recent studies (8-14) have explored the spatial and temporal variation of heterotrophic bacteria in water distribution systems. Correlations between Heterotrophic Plate Count or Standard Plate Count and turbidity, free chlorine residual, total chlorine residual and temperature have not been consistently proven. Correlations were sometimes positive, sometimes negative and no generalizations could be made that would fit data for all systems sampled. These results clearly indicate that bacterial contamination in distribution systems cannot simply be related to finished water quality. Even as early as 1936, Howard (15) attributed microbiological problems in distribution systems to "stirred up sediment in water mains particularly at dead ends as a result of varying water pressure and flushing...contamination of water in open service reservoirs,... accidental or deliberate introduction of foreign matter,... cross-connections and contamination caused by the use of new water mains...".

Geldreich *et al.* (13) reported that most bacteria enumerated by SPC from three water distribution systems belonged to the enteric gram negative group and could be placed into four categories of concern: total coliforms, coliform antagonists, opportunistic pathogens and a miscellaneous grouping of strains whose significance, if any, is not yet recognized. Coliforms can be introduced to treated drinking water from several sources. These organisms are not all indicative of a fecal contamination, rather, many total coliforms are naturally occurring in soil, water or plant material. One possibility is that some coliforms present in the source water may survive water treatment, incurring only sublethal and reversible injury (16, 17). Most sublethal stresses will result in damaged cells incapable of growing on commonly used enumeration media. These injured bacteria may, therefore, go undetected on the harsh culture media commonly used for coliform analysis. In the distribution system, or on home plumbing devices, under proper conditions of temperature and nutrients, injured organisms may repair cellular damage and become capable of multiplying. In dead end mains, for example, an active bacterial population may flourish in the planktonic phase. In addition an attached community of microorganisms, or biofilm, can develop on piping within the home and distribution system.

When all four monitoring programs described in this study were compared, routine macro-scale distribution monitoring program detected both the lowest incidence of TC-positive samples and the fewest E. coli. This would have been expected for several reasons. Firstly, the water sampled in the macro-scale program is fresher and any injured coliforms would have had little time to recover. Secondly, the water sampled in the macro-scale program would be exposed to fewer small distribution pipes and less likely to be contaminated by cross connections. Lastly, samples collected from the macro-scale monitoring are always collected after the screens or other point of use devices have been removed from the taps.

The results of the micro-scale monitoring were very similar to the sampling during water main repairs in terms of TC recovery, number of E. coli isolated, percentage of samples positive on resampling and species recovered. The chlorine residual detected in the micro-scale monitoring program (range of 0.11 mg/L to 0.41 mg/L, mean of 0.30 mg/L) was very similar to that detected in the macro-scale monitoring program (range of 0.16 mg/L to 0.38 mg/L, mean of 0.26 mg/L). The increased incidence of TC-positive samples in the micro-scale monitoring program cannot, therefore, be attributed to a lower free chlorine residual in the home samplings. Goshko et al. (18) speciated coliforms isolated from several distribution systems and reported that Enterobacter cloacae and Enterobacter agglomerans were found in samples with a free chlorine residual greater than 0.2 mg/L while E. coli were found only in samples without a free chlorine residual. In both micro-scale monitoring of private homes and sampling during water main repairs in Calgary, TC enumerated were also found to be primarily Enterobacter spp. However, as shown in Table 4, in the micro-scale monitoring E. coli was recovered from home samples with free chlorine residuals of 0.24 to 0.38 mg/L. There was no apparent relationship between free chlorine residual and the genera of total coliforms detected.

During the micro-scale monitoring program, no difference in the frequency of TC recovery between older and newer neighborhoods or between areas in the city receiving water from the Bearspaw or the Glenmore Water Treatment Plants were detected. Preliminary results of the home sampling program also indicate that the frequency of TC recovery did not show seasonal variation and was also unrelated to free chlorine residual and other water quality indicators. Many Total Coliform positive samples were recovered from home taps where the screens could not be removed.

The fourth set of data, collected during the coliform outbreak, was substantially different from all of the other three data sets in terms of TC, incidence of E. coli recovery, percentage of TC-positive re-samples and species recovered. The greatest numbers of TC, in the greatest frequency and with the lowest coliform species diversity were recovered during the coliform outbreak; the species most frequently recovered was E. coli.

TABLE 4

Total Coliform species identified and chlorine residual
present in 15 Total Coliform positive samples
collected during micro-sampling program

Date (D-M-Y) Sample Collection	Total Coliforms (CFU/100 mL)	SPECIES IDENTIFIED	Free Chlorine (mg/L)
21-12-1988	140	*Escherichia coli*	0.24
28-12-1988	9	*Enterobacter cloacae* *Escherichia coli*	0.25
03-01-1989	1	*Enterobacter agglomerans*	0.27
03-01-1989	11	*Klebsiella pneumoniae* *Enterobacter cloacae*	0.30
04-01-1989	5	*Escherichia coli* *Serratia odorifera* *Klebsiella pneumoniae* *Citrobacter amaloniticus*	0.38
04-01-1989	8	*Enterobacter cloacae*	0.38
04-01-1989	2	*Enterobacter cloacae* *Enterobacter edecarboxylata*	0.39
16-01-1989	2	*Enterobacter cloacae*	0.24
06-02-1989	1	*Enterobacter agglomerans*	0.38
20-02-1989	2	*Enterobacter cloacae*	0.35
27-02-1989	1	*Enterobacter cloacae*	0.32
02-03-1989	12	*Klebsiella pneumoniae*	0.33
24-04-1989	11	*Klebsiella oxytoca* *Enterobacter intermedium*	0.28
27-04-1989	2	*Enterobacter edecarboxylata*	0.25
27-04-1989	1	*Klebsiella oxytoca*	0.32

We know that the treatment processes at both the Glenmore and Bearspaw Treatment Plants were functioning well during the time of the unusual coliform isolations in the distribution system. Finished water quality coming from the treatment plants and water quality in the macro-distribution monitoring network showed no indication that coliform organisms were breaking through the treatment processes.

It is interesting to note that without exception, all of the TC isolated from a particular home during the coliform outbreak were the same species. In the micro-scale and water main repair monitoring programs, TC-positive samples frequently were a heterogeneous population of TC. Heterogeneous populations of TC are probably more typical of the normal distribution system flora, while TC plates covered with a homogeneous population may be indicative of a point source of bacterial contamination. During the coliform outbreak, re-samples were collected from the same home after the distribution mains had been thoroughly flushed with fresh water through the hydrants. These resamples sometimes, but not always, recovered the same coliform species as the original sampling. The presence of a second coliform species in the resample may simply have been normal distribution system flora and could have been unrelated to the original contamination. During the first few weeks of the coliform outbreak, many samples were collected from taps with screens still in place. These screens probably contributed many of the positive coliform isolates.

CONCLUSIONS

The data presented here represent preliminary results from an on-going study. Since the data set is still incomplete, it is not possible to definitively identify the source of total coliforms recovered in Calgary's distribution system. Perhaps it will never be possible to definitively pinpoint the cause of the transient and isolated incident of poor water quality in Calgary's distribution system in 1987. Antibiotic resistance testing and plasmid analysis, presently being conducted on all total coliform isolates, will provide more information on possible sources of these organisms. Further information is also required on the organisms associated with biofilm in Calgary's distribution system pipes and the recovery of injured coliforms following treatment.

No sampling network can statistically prove the complete absence of indicator bacteria throughout a water distribution system. Localized areas of the system may contain undetected pockets of bacteriologically poor quality water. Micro-scale and water main repair sampling programs both analyzed water which was more typical of that delivered to the consumers tap. The macro-scale monitoring program intentionally sampled large transmission and distribution mains to ensure the integrity of the system as a whole. If the TC incidence in home sampling is compared to that of the macro-scale monitoring program, it is easily possible to misinterpret the normally higher incidence of TC as indicative of a more serious water quality deterioration in the distribution system.

Quick response to taste and odour complaints is an important way to detect local contamination existing on a "micro-scale" within the "macro-scale" distribution system monitoring network. In Calgary, private homes are sampled expeditiously in response to taste and odour complaints for this reason. Much work (as described in Figure 1) remains to be done to provide a better understanding of bacteriological water quality in Calgary's large distribution system.

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DESALINATION OF BRACKISH GROUNDWATER FOR MELVILLE, SASKATCHEWAN

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BACKGROUND

Melville, Saskatchewan is a small city with a population of 5,200 located 150 km northeast of Regina. The City currently obtains its water supply from a reservoir on Crescent Creek about 3 km west of the City. Crescent Creek Reservoir has a full storage capacity of 4584 dam^3 and, based on its past hydrologic record, the reservoir is estimated to be capable of supply 421 dam^3 on a firm annual basis.

With the current annual demand of 850 dam^3, the City can expect to experience water shortages approximately 20 percent of the time. Crescent Creek Reservoir has not received significant inflow since 1985 which has resulted in chronic water shortages for the City. The City's annual demand is expected to increase to 1250 dam^3 by the year 2015. Figure 1 depicts recorded and projected per capita daily consumption and total annual demand for the City of Melville.

The quality of the water in the reservoir is considered acceptable with an average total dissolved solids (TDS) concentrating ranging from 400 to 700 mg/L.

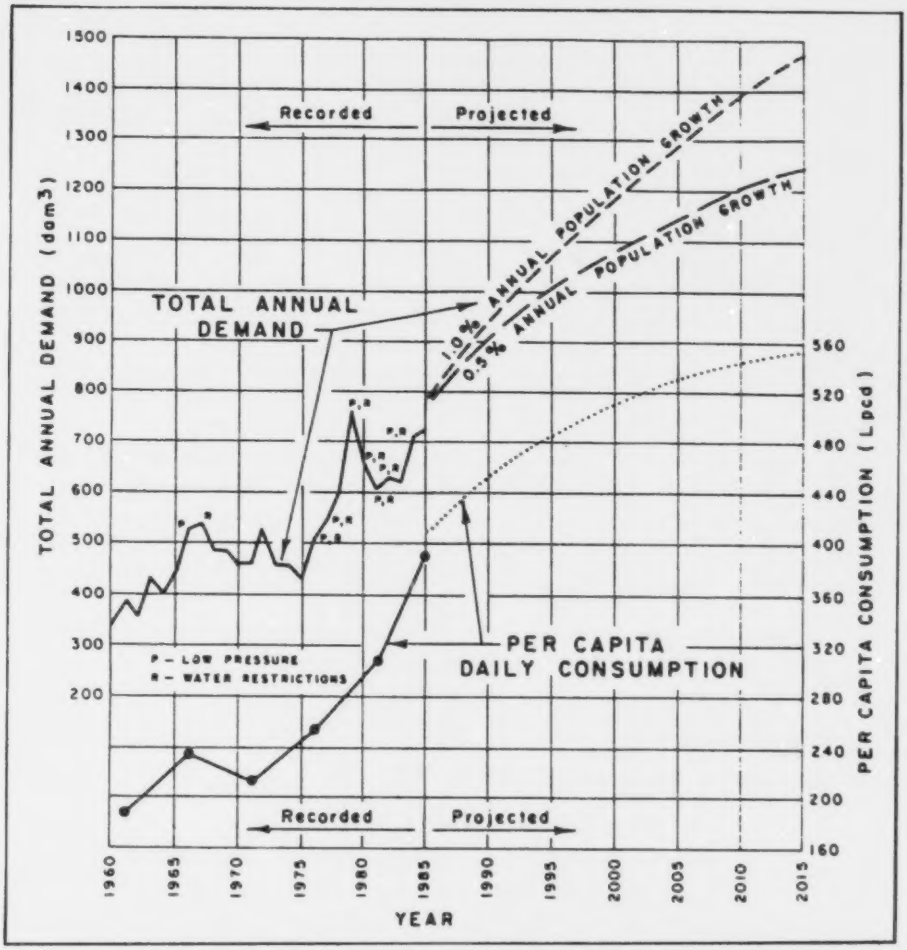
OPTIONS EXPLORED BY THE PRAIRIE FARM REHABILITATION ADMINISTRATION (PFRA)

PFRA conducted a comprehensive water supply study, completed in August 1986, under a Federal/Provincial agreement on Regional Economic Expansion and Drought Proofing. Several surface supply options were considered in the study, but these were discharged because of their poor water supply potential, geotechnical problems, or unacceptably high cost.

Groundwater in the vicinity of Melville is generally of poor quality when present in sufficient quantities for municipal purposes. As shown in Figure 2, the City of Melville is situated within the boundaries of the Hatfield Valley Aquifer System, an extensive but poor quality source; the average TDS concentration in the Melville area is 2200 mg/L with high sodium, sulphate, alkalinity, iron, and manganese concentrations. A cost-effective method of treating the brackish groundwater would make the aquifer a usable and drought proof supply for the City of Melville. The PFRA study recommended that groundwater be used to supplement the surface water supply, and that pilot studies be undertaken of a reverse osmosis desalination process to confirm its viability and to provide precise estimates of the capital and operating costs involved.

FIGURE 1

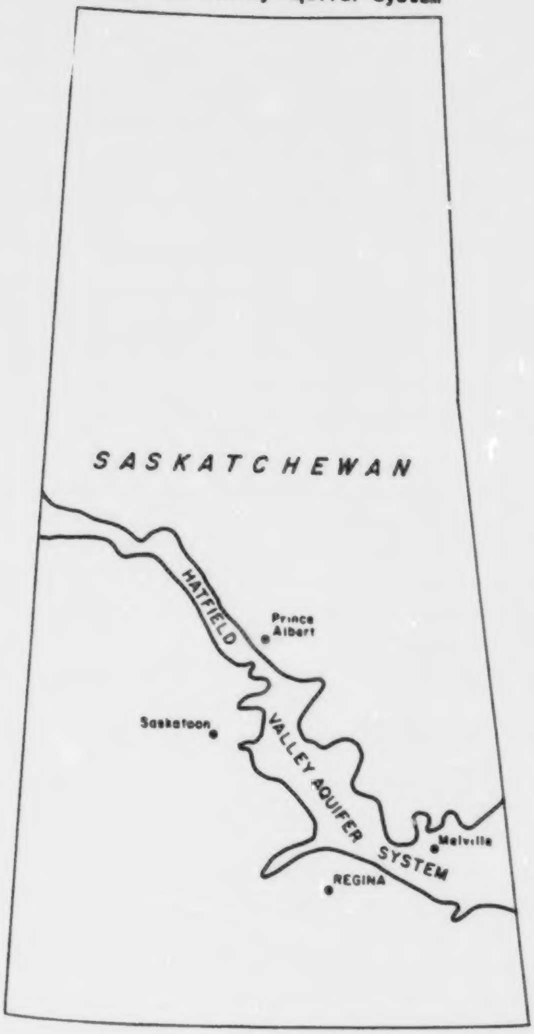
Per Capita Daily Consumption and Total Demand



CH2M Hill was subsequently engaged to undertake the pilot study. An examination of the water quality indicated that electrodialysis was a viable treatment option. Two desalination processes, reverse osmosis and electrodialysis reversal, were pilot tested to determine their suitability to the Melville groundwater and the Hatfield Valley Aquifer in general.

FIGURE 2

Hatfield Valley Aquifer System


SASKATCHEWAN
HATFIELD VALLEY AQUIFER SYSTEM
Prince Albert
Saskatoon
Melville
REGINA

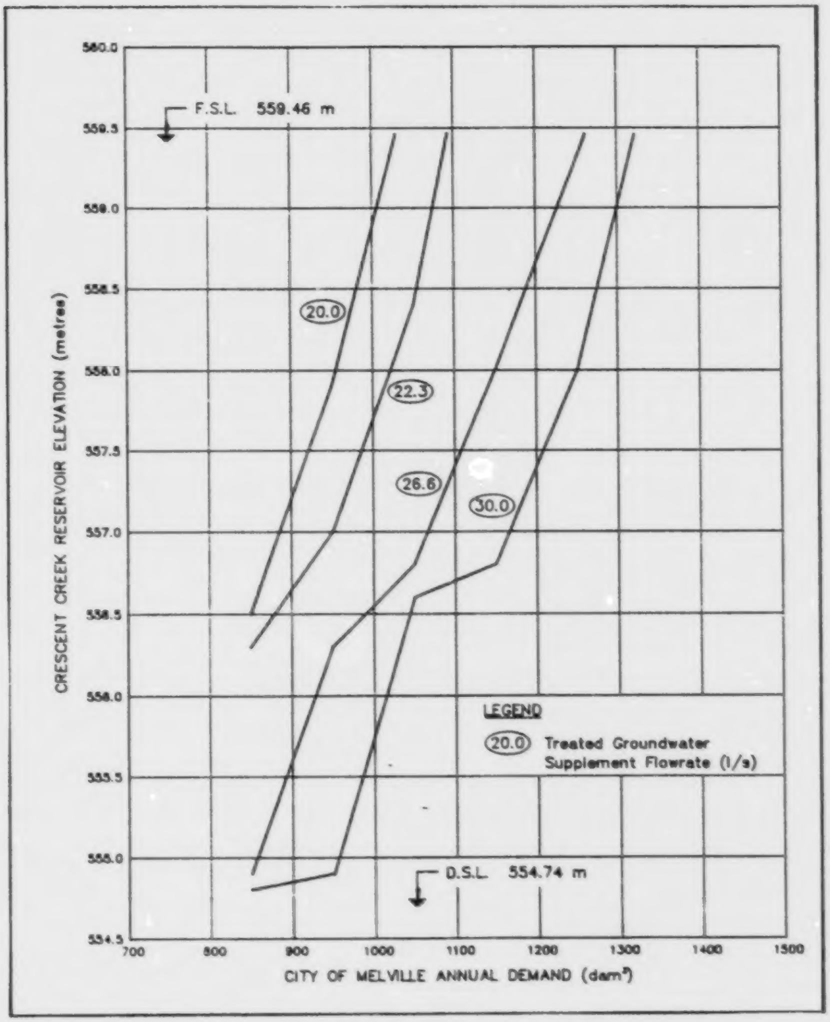
OPERATION OF A DESALINATION PLANT

For the City of Melville the desalinated groundwater would be used as a supplemental supply to the surface water supply to minimize operating costs. The desalination plant will only be operated when the Crescent Creek Reservoir level drops below a predetermined threshold elevation. PFRA developed a hydrologic model based on 46 years (1941-86) of hydrologic record for the Crescent Creek basin. The model estimates the reservoir elevation at which the desalination plant must be put into operation to avoid shortages in supply.

The threshold reservoir elevation is governed by the annual water demand and the size of the desalination plant. As shown in Figure 3, for annual demand of 1000 dam^3 and a desalination plant size of 20 L/s the corresponding threshold elevation would be 558.4 m in order to avoid shortages based on the hydrologic period of record. For the same demand and a supplemental supply of 22.3 L/s the threshold elevation would be 557.7 m. The threshold reservoir elevations will be used by the City to facilitate optimum operation of the desalination plant.

FIGURE 3

Crescent Creek Reservoir Threshold Levels
for Demand/Supplement Combinations



The model was also used to some extent in determining the optimum plant sizing, considering the higher capital costs of increased plant capacity versus the lower operating costs due to the shorter operating durations possible with larger production capacities.

REVERSE OSMOSIS (RO)

RO is the process of separating dissolved ions and other species from water by overcoming the osmotic pressure and forcing water through a semipermeable membrane from the stronger solution to the weaker solution.

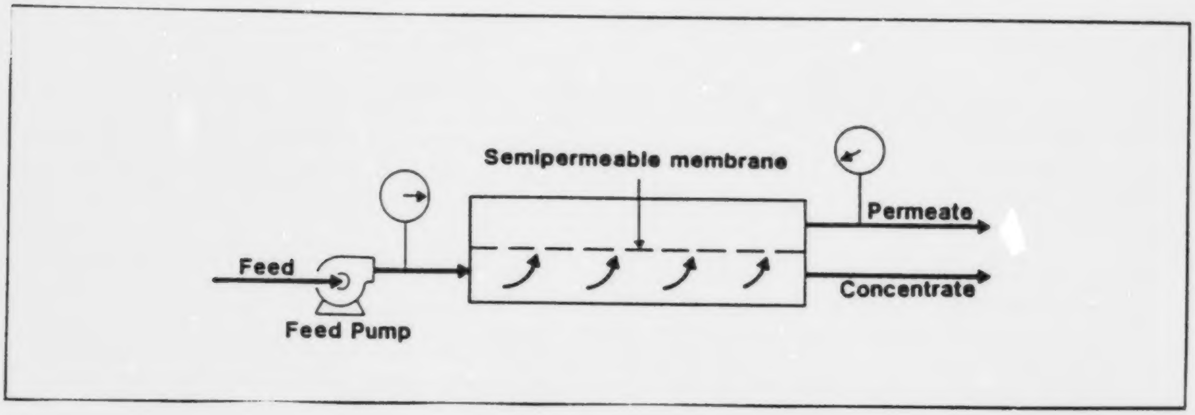
RO finds application in desalination of brackish water and sea water for potable use, as a first step in demineralization of industrial water, and, in an experimental way, in the removal of dissolved organics from contaminated groundwater.

Several materials are used for production of RO membranes. Cellulose acetate was the most common several years ago and still has application in certain systems. Recently, some high performance thin film composite membranes have been developed that exhibit improved water transport properties, which allows operation at lower pressure and higher flux (flow per unit area).

Figure 4 shows a simplified diagram of the RO process. A pump is used to raise the pressure on the feed side of the membrane. Water and a small amount of salt pass through the membrane. The majority of the salt remains in a concentrated brine stream that forms the concentrated reject flow. The operator controls the brine pressure and the recovery rate. The recovery rate is the fraction of feed that is recovered as product. Higher feed pressure results in a higher water flux rate but no change in salt passage. Higher product recovery results in a higher osmotic pressure and therefore a lower net driving pressure and lower water flux. It also results in a higher concentration driving force for the salt and increases salt passage.

FIGURE 4

Reverse Osmosis Schematic



Fouling of RO membranes is a major concern and results from three difference sources.

Suspended solids foul the membrane by being driven onto it with the water. Fouling occurs the fastest where the water flux is highest, at the upstream end of the membrane system. A highly efficient solids removal process is required as pretreatment for RO systems.

Biological growth will foul membranes if bacteria lodge in the surface of the membrane and proliferate there. If there are bacteria in the feedwater, it must be disinfected. This step is made more difficult if the membrane material is sensitive to chlorine, e.g. polyamide membranes.

Scale-forming salts will foul the membrane because the process of concentrating them often results in supersaturation in the brine stream. The degree of supersaturation is greater at the membrane surface. Since water is transported through the membrane but the salts are not, they must be carried away by diffusion. The salt concentration is necessarily higher at the membrane than in the bulk liquid.

Common scale-forming salts are calcium carbonate, calcium sulphate, barium sulphate and strontium sulphate. Techniques for scale control include limiting supersaturation by reducing recovery, pH adjustment, and addition of a scale inhibitor. Proper pretreatment is essential for efficient long-term operation of RO systems and can include filtration, iron removal, disinfection, and scale control. If fouling does occur, periodic cleaning with appropriate chemicals may be necessary.

The productivity or flux rate of water through the membrane is by far the greatest constraint on equipment size. Membrane configurations are therefore designed to maximize the membrane surface area to volume ratio. Several arrangements are available with spiral wound being the most common for brackish water applications.

RO membranes are sensitive to pressure surges which means that startup and shutdown should be as gradual and as infrequent as possible. When the system is shut down for more than a few hours, the brine side must be flushed with clean water to remove the supersaturated solution and prevent scaling. For longer shutdowns the membranes must also be filled with a preservative solution to inhibit biological growth.

ELECTRODIALYSIS

Electrodialysis is also a membrane process but is very different from RO. In electrodialysis the water flows unimpeded through channels in the membrane stacks while ions are carried by an electric current through anion and cation permeable membranes. The rejected salt is flushed away by a recirculating brine stream. Electrodialysis reversal (EDR) is a refinement of the basic electrodialysis process. This reversal helps reduce fouling problems.

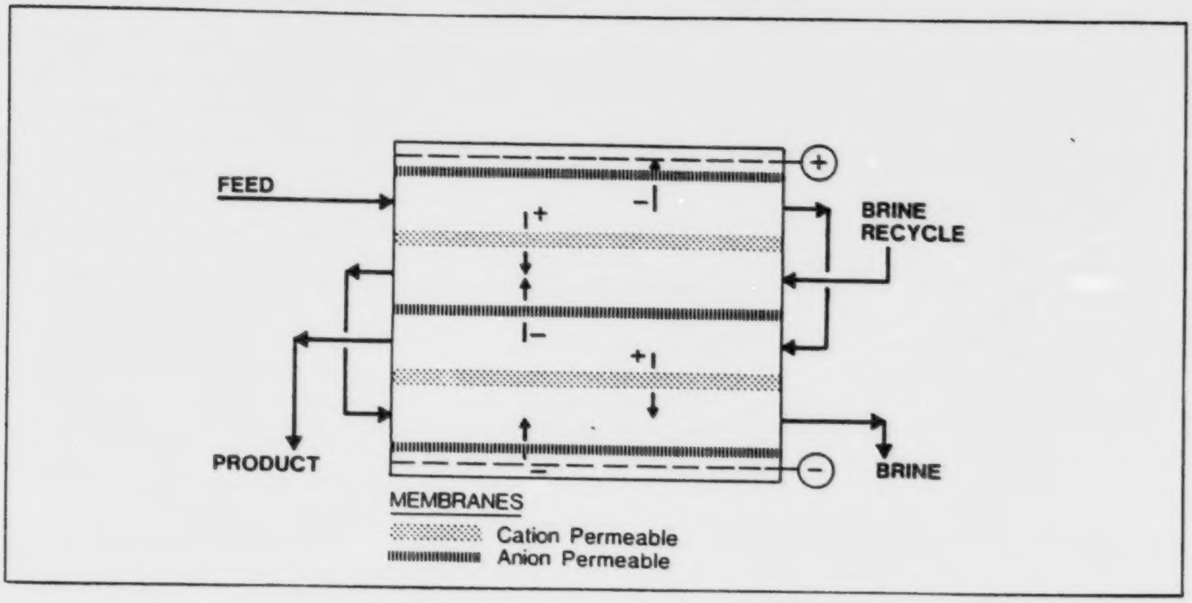
EDR is currently used primarily for desalination of seawater and brackish water for potable use and as a first step in demineralization of industrial water.

EDR membranes are made from essentially the same material used for ion exchange resins: anion permeable membranes from anion exchange resin material and cation permeable membranes from cation exchange resin material.

Figure 5 is a schematic diagram of the EDR process. Water is pumped through a serpentine passage between anion and cation membranes. Anion and cation membranes alternate in the stack as do feed and brine passages. When a DC voltage is applied at the electrodes, ions move under its influence through the membranes. Anions move from the feed cells through the anion permeable membranes to the brine cells. Cations move in the opposite direction from the feed cells through the cation permeable membranes to the brine cells. Since local electrical neutrality is required at all times, anions and cations must move in equivalent numbers (equal numbers of charges) out of any given feed cell and into any given brine cell. At intervals of approximately 20 minutes, external valves and the polarity of the applied voltage are switched to make the feed cell the brine cell and vice versa. There is a transition period during which the product water is high in TDS and must be wasted. The periodic reversal tends to reverse fouling tendencies caused by polarity concentration effects.

FIGURE 5

Electrodialysis Reversal Schematic



The hydraulic capacity of an EDR system is limited by the need, on the one hand, to maintain enough flow to ensure even distribution across the surface of the membranes, and on the other hand, to limit the inlet pressure to a value that the membrane stack can withstand. The salt removal capacity is limited by allowable current density and over-voltage. If the voltage is raised too high, other electrochemical reactions that are not desirable, such as reduction of chloride to chlorine, begin to occur. Product water quality is not as high as that produced by

RO. It is limited by the requirement for electrical conductivity in the desalinated water stream, which is a function of temperature, and by the high incremental cost of additional electrical stages. EDR product water TDS is generally between 100 and 500 mg/L, depending on the water temperature.

Fouling of EDR membranes occurs but is generally not as great a concern as it is in RO systems. Since the salt concentration in the brine cells increase by transport of ions into the cells instead of transport of water out of the cells, the concentration gradient is much lower than in an RO system. Since only one of the two ions that comprise the salt passes through any given membrane, the highest salt concentration is in the bulk liquid, not at the membrane surface. Periodic reversal of the direction the current travels also helps reverse fouling reactions that do occur.

The result is that EDR can tolerate a higher degree of supersaturation in the reject stream than RO. Antiscalant is added to the brine recycle stream in an EDR system rather than to the feed stream as in an RO system so that a much smaller amount is required. Side reactions occur in the electrode compartments because they are one-sided cells with only an anion membrane or only a cation membrane boundary. A small flow of water is directed through the electrode compartments to flush away the acidic and basic solutions generated there.

Startup, shutdown and storage of an EDR system are relatively simple. For extended storage the membranes must be kept wet. Periodic cleaning of the membranes and electrodes is part of normal operation. Since the membrane material is very durable, high strength, aggressive cleaning solutions can be used without concern.

HATFIELD VALLEY AQUIFER

Feedwater quality plays an important part in the design and operation of membrane desalination systems. A summary of the water composition in the Hatfield Valley aquifer in the Melville area is shown in Table 1. Water from other areas of the aquifer is similar in composition.

TABLE 1
Hatfield Valley Water Quality

Parameter	Concentration	Parameter	Concentration
Calcium	180	Sulphate	729
Magnesium	90	Chloride	29
Sodium	250	Bicarbonate	580
Potassium	<1	Nitrate	<0.05
Iron	1.4	Nitrite	<0.01
Manganese	0.5	Fluoride	<0.01
Barium	0.01	Silicon	12
Strontium	1.3	TDS	1620
Aluminum	<0.05	TSS	0
pH	7.1	Conductivity	2150 µS/cm
SDI	0.3 - 0.5	TOC	6.7

All concentrations are in mg/L

Review of the water composition led to identification of the following scaling concerns relative to RO operation and their impact on pretreatment requirements.

Calcium carbonate: At a recovery rate of 75 percent, acid addition is necessary to prevent carbonate scale from forming on the membranes. A feedwater pH of 6.5 was selected for the initial value.

Calcium sulphate: Sulphate scale is a limiting factor in this water. Scale inhibitor is required to achieve a recovery rate of 75 percent.

Barium sulphate: The concentration of barium sulphate is so low that precipitation is not a concern in this water.

Strontium sulphate: The strontium sulphate concentration is less than 50 percent of saturation at 75 percent recovery.

Iron: The Iron concentration is relatively high. If there were any oxygen present in the water or if it were to be chlorinated, iron fouling would be a serious concern. Since neither situation was encountered, iron removal was not considered necessary.

Bacteria: Initial tests indicated the presence of bacteria. A second test after pumping the well for period of time showed no bacteria. Disinfection was not required.

Suspended Solids: After a period of flushing the well, the SDI was measured at less than 10. During the pilot test it declined further to approximately 1.

Silica: At 75 percent recovery, the silica concentration in the brine stream is slightly higher than saturation which will result in slow fouling of the final membrane elements. Since silica removal is very expensive, a low rate of fouling was considered acceptable.

The pretreatment requirements for EDR operation are generally less demanding than for RO. However, concerns identified and the means of dealing with them are as follows:

Iron and manganese: EDR is sensitive to iron and manganese even in the absence of oxygen because of the electrochemical oxidation within the unit. Greensand filtration was specified for EDR pretreatment.

Product water quality: At the temperature of the source well, the electrical conductivity of the product water and the design staging of the EDR unit limit the salt removal to approximately 500 mg/L. This met the product water objective set out at the beginning of the project which was based on the quality of the surface water supply.

PILOT TESTING PROGRAM

In order to confirm the preliminary design parameters and to obtain operating experience with the two processes, two pilot plants were assembled at the Melville water treatment plant. Water was drawn from a new source well and pumped to the plants through existing pipelines. Major components of the pilot plants were:

RO Pilot:

Pretreatment: Acid and antiscalant addition, 1 cartridge filtration.

RO System: UOP Model 4821LP TCFL membranes, 18 4" membranes in a 4-2 array.

EDR Pilot:

Pretreatment: Chlorination, permanganate addition and manganese greensand filtration for iron and manganese removal, 10 u cartridge filtration, antiscalant addition to brine stream.

EDR System: Ionics Aquamite V trailer-mounted EDR system.

All waste and product streams were sent to the sewer. City of Melville water treatment plant staff carried out all operating and maintenance functions and the analytical work that could be done onsite. Other analytical work was done by a commercial laboratory.

Operation of the pilot plants was routine and simple. RO pilot operations, including data collection, sample collection and analysis, and chemical applications generally took 2-4 hours per day. Equivalent tasks for the EDR pilot took 1.5-2 hours per day.

RO Pilot Program

The initial operating conditions for the RO system were set based on performance calculations carried out independently by CH2M Hill and the membrane manufacturer. Initial conditions were:

Recovery	75%
Feed pH	6.5
Antiscalant	5 mg/L Flocon 100
Expected Flux:	12.5 gfd [gallons/ft^2/day]
Expected Flow:	12 gpm

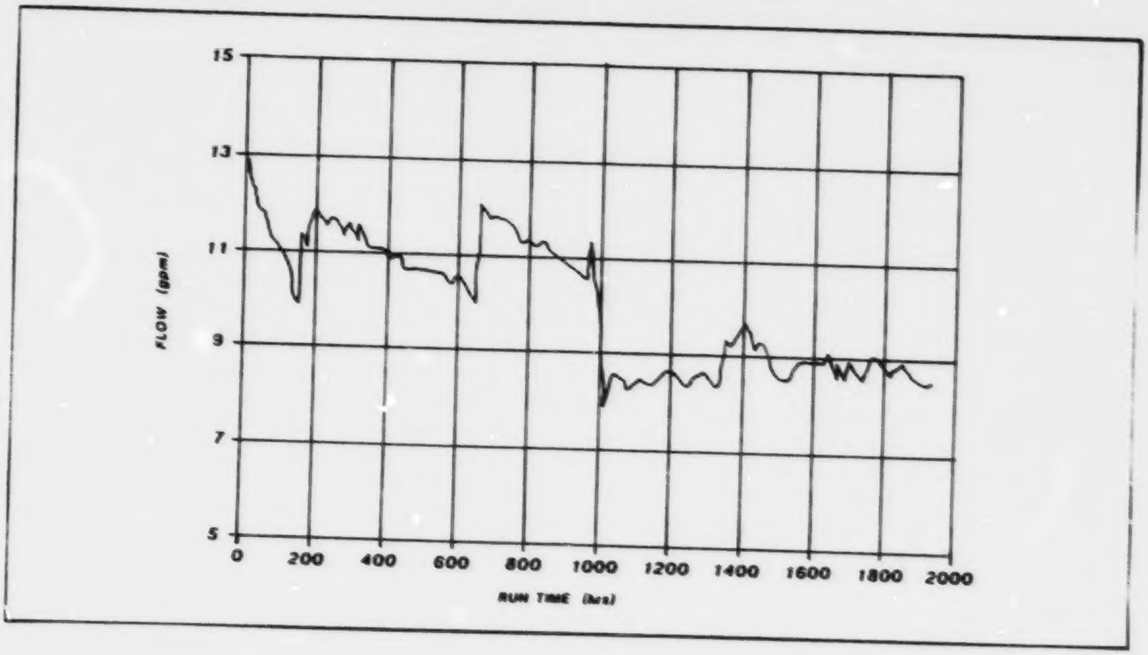
Product water TDS was approximately 50 mg/L at the start of the run and improved to approximately 30 mg/L over the first 1,000 hours of operation. Rejection of monovalent ions was 96-98 percent and rejection of divalent ions was 98-99 percent. In addition to removal of inorganics, a 70-90 percent removal of TOC was observed.

Actual normalized flow over the first 2,000 hours of operation is shown in Figure 6. Measured RO flow-rates and salt passage were normalized to a feed pressure of 1605 kPa and a temperature of 10°C to allow comparison of data.

The initial flow of 10 gpm did not meet the manufacturer's performance projection. Also, as can be seen from the decline in flux at the beginning of the test, the membrane was rapidly fouling from the start of the test. The membranes were cleaned once with Floclean IPA-403. Analysis of the cleaning solution showed the presence of iron, calcium, zinc, and silicon on the membrane. Cleaning improved the flux but it continued to decline, although at a slightly lower rate. Another cleaning, this time with citric acid, was carried out and the flux was again restored to startup conditions.

A third cleaning with a combination of an alkaline cleaning compound and formaldehyde resulted in an irreversible reduction in flow to approximately 8.5 gpm. Operation after the third cleaning at a reduced recovery and feed pH was stable.

FIGURE 8
RO Pilot Plant Normalized Product Flow



Review of the operating data indicates that three separate problems occurred which were not consistent with any single cause:

1. The flux rate never achieved the predicted value, even on the first day of operation.

2. Membrane fouling occurred consistently at conditions that performance calculations indicated were satisfactory. Iron and zinc appeared to be the chief constituents of the fouling material.

3. Irreversible damage was caused to the membrane by the use of an alkaline cleaning compound with formaldehyde which was not associated with any material remaining on the membrane.

Subsequent stable operation at reduced recovery and flux provided some assurance that the recurrent fouling could be avoided. The combination of an alkaline cleaner and formaldehyde would not be used again. It was concluded that stable operation at 8 gpm could very likely be achieved.

New membranes were obtained and put in service for an additional 1,000 hours. This follow-up test showed that not only could 8 gpm be achieved, but that stable operation at 12.5 gpm and 75 percent recovery could be achieved. During that time the recovery was increased from 70 to 75 percent and the feed pH was increased from 6.0 to 7.0. During the whole test the standardized flux was 13 gfd.

The observation that the water flux reached the predicted value not only on the first day, but for an extended period supports the supposition that the original membranes were not perfect when they were received. The only difference in operation that would account for the lack of fouling seen in the supplemental test is the use of a different antiscalant.

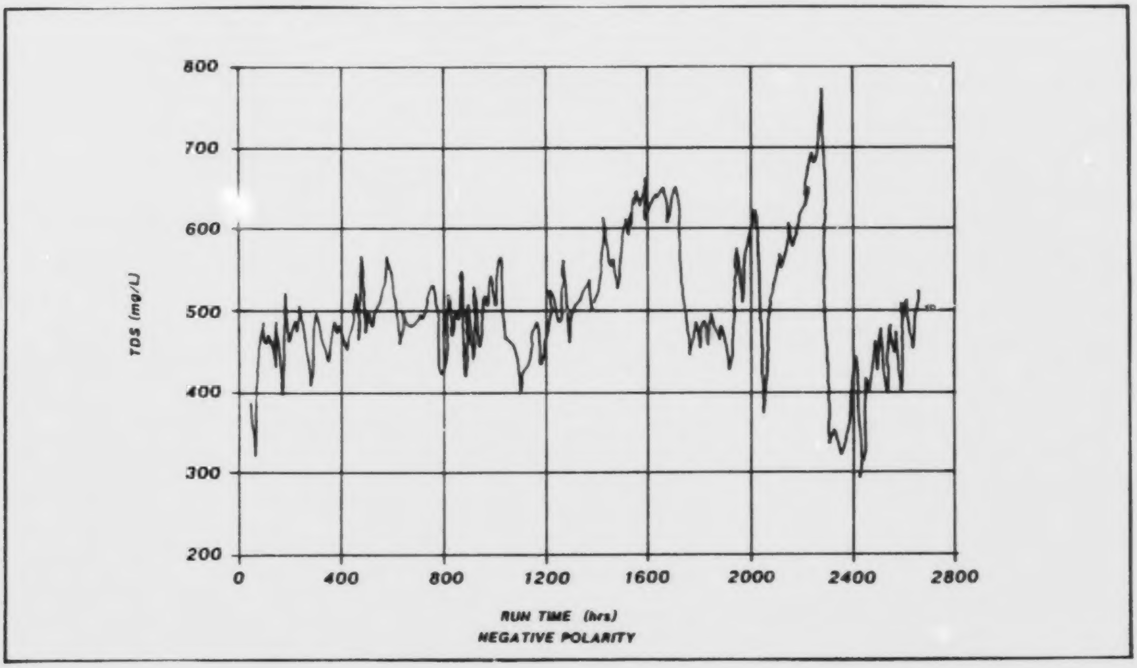
EDR Pilot Program

The EDR pilot was operated for approximately 2,500 hours at the following conditions:

Feed flow: 13 gpm
Recovery: 85%

The key performance parameter of the EDR system was the product water TDS. Even if the rectifier system was totally non-functional, the feed and brine flows would remain the same since they depend only on pump and valve action. Figure 7 shows the product water quality over the period of pilot operation. Separate curves were constructed for positive polarity and negative polarity operation. The two curves were essential the same.

FIGURE 7
EDR Pilot Plant Product Water Quality



Three episodes of increasing product water TDS occurred. The first was due to a fouling episode that developed in one of the electrode compartments. It was cleaned but the cleaning procedure was not clear and the cleaning was not thorough. A second decline in product water TDS occurred for the same reason and it was cleaned again. The third time the fouling occurred it was determined that one of the electrodes (which had been in service for a number of tests) had reached the end of its normal life and it was replaced. In each case it was evident that normal operating procedures would be effective in preventing this type of problem in a full-scale plant.

COST COMPARISONS

Preliminary cost estimates were prepared for the two processes. Each system was based on a desalination capacity of 22.3 L/s of product water with a TDS specification of 500 mg/L of TDS. In the case of RO this meant that approximately 23 percent of the product water bypassed the RO unit to blend the TDS upward from the RO permeate concentration.

A summary of the capital and operating costs for the two systems is shown in Table 2. Facilities included that are common to both estimates include source well, feedwater supply pipeline, and brine disposal facilities. Equipment included in the RO system consists of acid and antiscalant feed systems, iron removal equipment, dechlorination equipment, the RO modules, a clean-in-place (CIP) system, a decarbonator, and associated utilities. Equipment included in the EDR estimate consists of chlorination, manganese greensand filtration, the EDR package, CIP systems, and associated utilities.

TABLE 2
Estimated Desalting Costs

	RO	EDR
Desalting unit feed flow (L/s)	23.9	26.5
Desalting unit product flow (L/s)	16.7	22.3
Blended product flow (L/s)	22.3	22.3
TOTAL CAPITAL COST ($1,000)	1,965	1,942
OPERATING COSTS ($\$/\text{m}^3$ of product)		
Electricity (4.9c/kWh)	0.11	0.10
Chemicals	0.12	0.02
Membrane Replacement	0.08	0.06
Labour (O&M)	0.04	0.03
Other Consumables	0.03	0.03
TOTAL	0.29	0.24

Intermittent operation, as planned by the City of Melville, would further favour EDR because of the lower effort involved in startup and shutdown compared to RO.

These costs were prepared before the supplemental RO testing and include iron removal equipment for the RO system because of the indication at that point the iron was involved in fouling the RO membranes. The supplemental testing indicates that iron removal should not be necessary so that the cost of an RO system would be somewhat lower than shown above.

CONCLUSIONS

The first and most important conclusion from this work is that both RO and EDR are suitable desalination processes for Hatfield Valley water. There are cost, waste rate, pretreatment, post-treatment and operability considerations that will affect the choice but there are no basic concerns that will prevent their use.

A second conclusion is that pilot testing is a valuable exercise in selecting a desalination process. Apart from design information being established, there are problems that arise and can be addressed that cannot be predicted by current performance calculations. In the case of RO, the cost of a ruined set of membranes in a full-scale plant would more than offset the entire cost of the pilot program.

The third conclusion to be drawn is that EDR is much easier and more forgiving to operate. There are fewer opportunities to make mistakes and the consequences are much less severe than for RO. EDR is also more tolerant of intermittent operation and less demanding in storage requirements than RO. This was an important consideration for the City of Melville because the desalination process will be operated intermittently as a supplemental supply.

The total installed costs for the two systems treating Hatfield Valley water are very close. EDR equipment is more expensive than RO equipment, but the chemical, membrane replacement and post-treatment costs make the total costs more nearly equal.

IMPLEMENTATION

As a result of this study, the City of Melville has elected to install an EDR system of 22.3-L/S capacity as a supplemental water supply system. Startup is scheduled for March 1990.

AUTOMATIC COMPUTER CONTROL
AND THE
UTILITY OPERATOR

THE CITY OF CAMROSE EXPERIENCE

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Assistant City Engineer
City of Camrose

INTRODUCTION

Computer Control systems have been in service in one form or another for decades. The improvement in these systems over the past ten or so years has been remarkable. Unfortunately the water and waste water industries have tended to fall behind other industries such as the manufacturing and oil field industries in their use of these systems.

In may situations Computer control is overlooked because design engineers or local operators have the impression that they are too complex for the local staff to operate or maintain. In other situations, computer control is installed but not used to its full potential, because the project engineers, or local operators were not aware of the full potential of a computer system, and therefore, could not give adequate instructions to the programmer.

The City of Camrose has been utilizing Programmable Logic Controllers (PLC's) for pump station control since 1983 when High Lift Pumping Station Number Three, a new reservoir and pumping station, was constructed. Since that time all six Camrose water pumping stations have been upgraded to PLC control.

All City of Camrose Water Pumping stations run entirely in an automatic mode and are not manned. The PLC's not only control the stations, but communicate between each other allowing central reporting of equipment status, alarms, and flow summaries.

When the first system was installed in 1983, none of the Camrose staff had any experience either programming or operating PLC's. Unfortunately, the PLC program which was designed for the station did not allow the operator any monitoring, or control features. The program was designed to run the station in automatic mode only, with no allowance for manual override, no allowance for data collection, very limited reporting, and no easy way for the operator to change setpoints.

This situation was not acceptable, so after taking several PLC programming courses, the entire program was re-written by the author.

All of the PLC programming for the City of Camrose since 1983 has been done entirely in house, and we have, therefore, been able to tailor each system to meet the requirements of our operators.

This paper is written to familiarize the reader with modern PLC operations, explain some of the features available with PLC/Computer Control, and to describe how a PLC and computer monitoring system was successfully installed at the new Camrose Water Treatment Plant, which was constructed in 1987.

DESCRIPTION OF A COMPUTER CONTROLLED SYSTEM

Typically, a computer controlled process can be divided into four components:

1) Plant Process

The plant process is the physical system to be controlled. It includes valves, motors, flow meters, pressure meters, Ph meters, Turbidity meters, or other systems.

2) Input/Output System

The input/output system is a collection of electrical connections between the plant process, and the Programmable Logic Controller (PLC).

There are two basic forms of input, and two of output; digital and analog:

A digital input is an off/on contract such as a switch, or motor status contact.

A digital output is an on/off signal which starts or stops a system such as a pump, valve, or indicator lamp.

An analog input is a variable input signal, usually a 4 to 20 ma signal, which represents a process variable such as a flow or pressure measurement.

An analog output is a variable output signal, usually 4 to 20 ma, which controls a variable process such as a Variable Speed Motor Drive, or Control Valve.

Note that the plant process and Input/Output system are common to either a computer controlled system, or a more traditional hard wired controller system.

3) Programmable Logic Controller (PLC)

The programmable Logic Controller (PLC) is a very fast computer designed specifically to connect to a plant process via an input/output system. A PLC receives signals from the process, in both digital and analog format, makes decisions based on program logic, and then sends controlling signals back to the process.

As a General rule, PLC's are programmed in a language called "ladder logic", which is a graphically based program designed to look and read in a similar manner to an electrical wiring diagram.

A very simple example of a ladder logic program is shown in Figure 1.

This one line program monitors the status of "SWITCH ONE", (connected to terminal 07 of an input module located in rack 1/group 4), and turns on "MOTOR ONE" (connected to terminal 15 of an output module located in rack one group 8) when the switch is turned on, or turns the motor off if the switch is turned off.

One of the big advantages of PLC control is its inherent flexibility. If, for example, a one minute delay was required after "SWITCH ONE" is turned on, before "MOTOR ONE" starts, a simple program change is all that would be required:

Figure 2 shows an example of a program which would start "MOTOR ONE", 60 seconds after "SWITCH ONE" is closed. Note that no physical change to either the wiring or progress was required.

FIGURE 1
A PLC Program To Control "Motor One"

```
      SWITCH ONE                                    MOTOR ONE
        114                                           014
------ ||------------------------------------------(    )---
        07                                             15
```

FIGURE 2
A PLC Program To Control Motor One With A 60 Second Delay When Started

```
SWITCH ONE
                                         timer     60 second
 114                                      400        timer -
---||-----------------------------------(TON)---  counts in
 07                                       1.0     one second
                                          000        units
                                          060


 time out
                                        MOTOR ONE
 400                                      014
---11-----------------------------------( )---
 15                                       15
```

PLC Programs can be designed to control any type of process using combinations of commands such as timers, counters, control outputs, normally open contacts, normally closed contacts, control latch and unlatch statements, math functions, PID Functions, logic functions, and others.

While programs can be several thousand lines in length, and very complicated, individual segments of the programs controlling individual components are usually fairly simple.

Because PLC programs are difficult to read, especially for complex functions, and because operators are generally not programming experts, it is essential that any PLC program be fully documented in English. The program documentation must explain in English exactly how each process or alarm functions and must explain all of the conditions which could affect the alarm and process. There is nothing which frustrates an operator more than a PLC which does something he does not expect it to do. Even if the operator was involved in the initial programming process, it is easy to forget all of the criteria included in the programming logic for each alarm or process.

For example; assume that in the initial programming of a station, it is decided that a water pump should shut down if a particular valve fails to open.

Several years later, if a problem develops with the contact in the valve, and the PLC does not receive the Valve Open signal and shuts down the pump, without adequate documentation describing the complete pump control, the operator may not relate the pump failure to the failure of Valve Open contact.

4] Operator Interface - Control/Monitoring Station

While the PLC does an excellent job of controlling the process, it is not in itself a very user friendly device. While it would be possible for an experienced programmer to connect a terminal to the PLC and view or modify setpoints or control variables, etc. directly in the PLC Ladder Logic Program, it is an awkward process, and not easily accomplished by the average individual.

It is, therefore, necessary to attach some form of operator interface to the PLC to allow the operator easy access to view important variables, change setpoints, and control operations. In the past, this have been accomplished by the use of large control panels with many switches, readouts, dials, and charts.

A computer/monitoring station which is usually connected to the PLC by a single communication cable, is able to read or control variable, setpoint, control bit, or status list within the PLC memory.

These systems, if programmed properly, are very simple to operate. The commands are in English, setpoints are in true units, and the units are shown on the display. Any system or variable controlled by the PLC can be displayed and controlled from the monitoring station and keyboard.

Because the computer monitoring/control station requires only one cable for connection to the PLC, it is very easy to relocate, or add additional stations at any location within the Plant. In the Camrose Water Treatment Plant, a remote terminal was installed adjacent to the filters to monitor and control filter operations.

In addition, printed reports can be automatically generated based on data collected by the PLC. These can include hourly status summary reports, daily summaries, alarm or status summaries, and others. Most systems also offer trending, [similar to an electronic chart recorder], and other options.

During the past five or so years, computer monitoring/control stations have improved a great deal, and the cost has dropped dramatically.

It is now possible to purchase a computer monitoring/control station which provides graphic capability, and virtually unlimited access to PLC variables, setpoints and control parameters, for less money than the cost of a conventional control panel.

CAMROSE WATER TREATMENT PLANT

When the Camrose Water Treatment Plant was constructed in 1987/88, it was decided to involve the chief operator in the design and programing process from the start to ensure that the system was user friendly, and met all his requirements.

In this case the Water Treatment Plant operator had little or no experience with computer systems or PLC control. By involving him in the design and programming process from the start, we were able to work out many operational problems, and familiarize him with PLC operations prior to commissioning.

As with an PLC installation, all of the mechanical, electrical, and instrumentation work has to be essentially complete before the PLC can be tested. Of course, by that time commissioning, is at hand so the program is expected to be installed and tested within a very limited time span.

The first step in programming the PLC was to meet with the operator, and review all of the control features, and alarms required. Each system was discussed to determine the exact control sequence, and logic that would be required by the PLC, and what control and monitoring was required by the operator.

A skeleton program was then installed and the Plant was commissioned with a combination of manual and automatic control.

Each system was then looked at again in detail and the program enhanced to meet with the operator's requirements. Some of the more complicated systems, such as the filter automatic backwash and chemical feed systems were installed in several stages:.

Filter Control and Backwash

The first stage of development of the backwash system was to institute a manual control system so that the operator could monitor filter operation, and manually backwash the filters, controlling valves and pumps from a remote computer terminal located adjacent to the filters.

After several weeks of manual operation, a manually initiated automatic backwash system was installed so that the operator could backwash a filter simply by initiating a backwash instruction on the terminal.

Again the system was tested for several weeks, and then, after further review and discussion with the operator, a fully automated system was implemented, whereby the PLC monitors all filter processes and initiates a backwash automatically if certain conditions are met.

Some of the criteria for backwash are as follows:

Filter Time	> setpoint	72 hrs
Filter Headloss	> setpoint	3.5 m
Filter Turbidity	> setpoint	0.5 ntu
Filter Effluent Valve	> setpoint	90 %
Filter Production	> setpoint	6 ML

Other conditions include:

No filter backwash may be initiated if the clearwell level is less than a low setpoint.

No filter will commence backwashing until at least two (setpoint) hours after the previous backwash.

No backwash will be initiated if the centrifuge is running.

No filter will be allowed to backwash on Turbidity unless the filter has already been in operation for a minimum of five (setpoint) hours.

Upon completion of each automatic backwash, a report is printed which describes the reason for the backwash, as well as the filter parameters before and during the backwash.

While the system is operated in a fully automatic mode 98% of the time, the program is designed so that the operator still has the option of either initiating an automatic backwash manually, or manually controlling any valve or pump via the remote computer terminal.

Chemical Feed Systems

The operator was again consulted to develop the logic and control systems for each chemical feed system.

Upon commissioning, a skeleton system was installed which was then modified over the next few months to meet the operator's requirements.

After some discussion, it was decided to allow the operator to set chemical dosages in the desired units directly (mg/l), rather than having to continually make conversions based on pump speed, flow, etc.

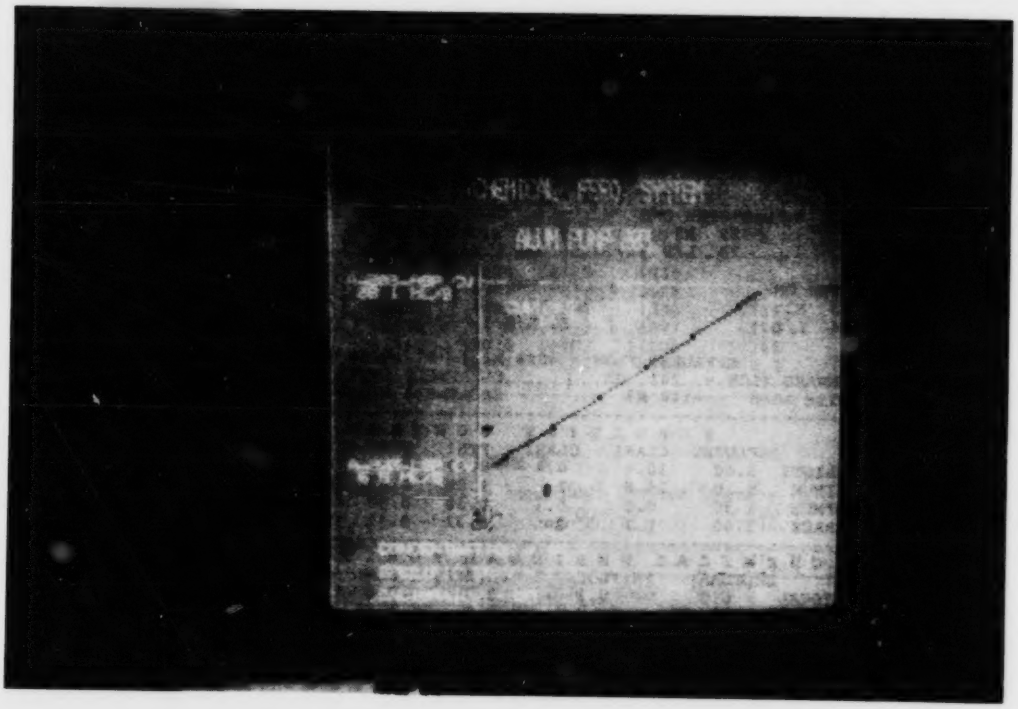
To do this, it was first necessary to develop a metering pump calibration screen so that pump calibration data, as well as chemical concentration and specific gravity, could be input to the PLC. A typical calibration screen is sown in Picture 1.

The operator calls up the calibration screen for the specific pump he is calibrating, sets the pump to maximum speed, and then measures the amount of chemical the pump is pumping. He then sets the pump to minimum speed, and again measures the amount of chemical the pump is pumping.

The measurements are then input as the pump maximum, and minimum outputs in the actual units used, (ml/s for liquids, and kg/min for powders). The operator then inputs the chemical concentration and specific gravity. Once this is accomplished, the operator is required only to input the desired chemical dosage, in mg/l, and the PLC is able to calculate the proper pump output based on the current flow, the pumping range of the metering pump, and the specific gravity and concentration of the chemical.

The PLC constantly monitors the process flow and varies the metering pump output to maintain a correct chemical dosage.

PICTURE 1
Chemical Pump Calibration Screen



The final stage of program development at the Camrose Plant was finalization of report generation. Again the operator was consulted to determine exactly what information he required, and in what format.

As with the other portions of the program, the reports were developed in stages, with a skeleton report developed first which contained critical information, and then enhancements added later.

Several printed reports are now generated using two printers, and one phone-in report is available by using a portable computer to phone the PLC at the Water Treatment Plant.

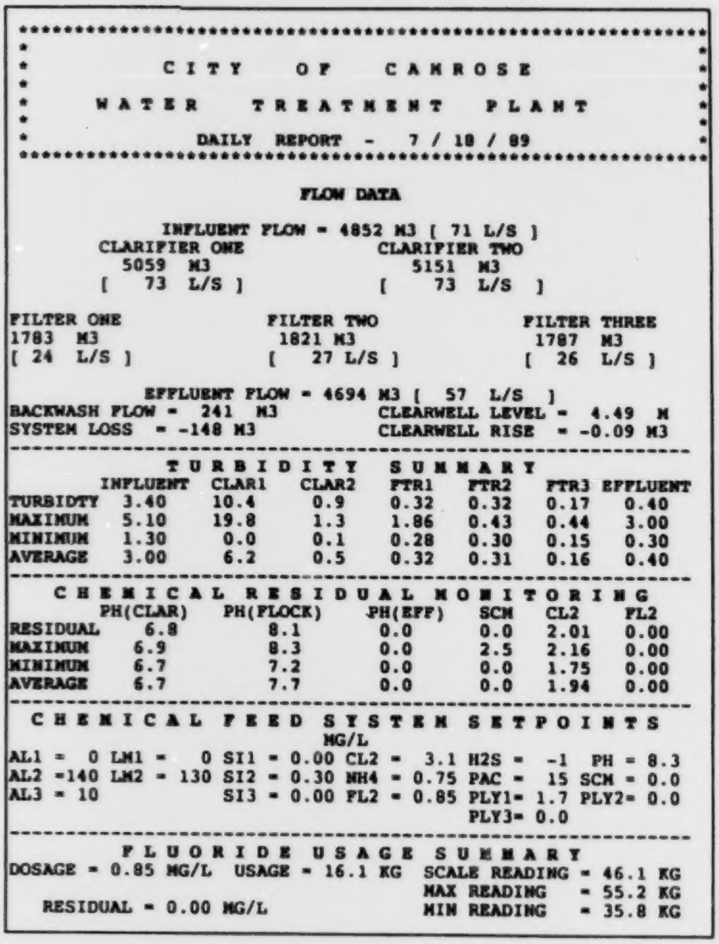
PRINTER ONE

1) **Daily Report - (Figure 3)** - summarizes current and total daily flows, reservoir levels, maximum, minimum, and average Turbidities, maximum, minimum and average chemical residuals, as well as the Chemical Setpoints for each metering system as of midnight. The report also contains special section which summarizes fluoride usage.

2) **Consumption History Report - (Figure 4)** - Summarizes supply flow/ distribution flow/Water Treatment Plant production, and distribution reservoir levels during the past five days and estimates the current day consumption based on a simple linear approximation formula. This report gives the operator an idea of how fast the reservoir levels are rising or falling and assists him in setting the Plant flow.

FIGURE 3

Water Treatment Plant Daily Report



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*************************************************************
*                                                           *
*           C I T Y     O F     C A M R O S E               *
*                                                           *
*    W A T E R    T R E A T M E N T    P L A N T            *
*                                                           *
*            DAILY  REPORT  -   7 / 18 / 89                 *
*************************************************************

                        FLOW DATA

           INFLUENT FLOW = 4852 M3 [ 71 L/S ]
      CLARIFIER ONE                 CLARIFIER TWO
        5059  M3                        5151  M3
      [   73  L/S ]                 [     73  L/S  ]

FILTER ONE                FILTER TWO               FILTER THREE
1783  M3                   1821 M3                   1787  M3
[ 24  L/S ]               [    27 L/S ]            [  26  L/S ]

          EFFLUENT FLOW = 4694 M3 [  57  L/S  ]
BACKWASH FLOW =  241  M3            CLEARWELL LEVEL =  4.49  M
SYSTEM LOSS  = -148 M3              CLEARWELL RISE  = -0.09 M3
--------------------------------------------------------------
             T U R B I D I T Y   S U M M A R Y
        INFLUENT  CLAR1    CLAR2    FTR1   FTR2   FTR3 EFFLUENT
TURBIDTY  3.40    10.4      0.9     0.32   0.32   0.17   0.40
MAXIMUM   5.10    19.8      1.3     1.86   0.43   0.44   3.00
MINIMUM   1.30     0.0      0.1     0.28   0.30   0.15   0.30
AVERAGE   3.00     6.2      0.5     0.32   0.31   0.16   0.40
--------------------------------------------------------------
    C H E M I C A L  R E S I D U A L  M O N I T O R I N G
        PH(CLAR)    PH(FLOCK)    PH(EFF)    SCM    CL2     FL2
RESIDUAL    6.8        8.1         0.0       0.0   2.01    0.00
MAXIMUM    6.9         8.3         0.0       2.5   2.16    0.00
MINIMUM    6.7         7.2         0.0       0.0   1.75    0.00
AVERAGE    6.7         7.7         0.0       0.0   1.94    0.00
--------------------------------------------------------------
  C H E M I C A L  F E E D  S Y S T E M  S E T P O I N T S
                            MG/L
AL1 =  0 LM1 =    0 SI1 = 0.00 CL2 =  3.1 H2S =  -1  PH = 8.3
AL2 =140 LM2 = 130 SI2 = 0.30 NH4 = 0.75 PAC =  15 SCM = 0.0
AL3 = 10           SI3 = 0.00 FL2 = 0.85 PLY1= 1.7 PLY2= 0.0
                                         PLY3= 0.0
--------------------------------------------------------------
        F L U O R I D E  U S A G E  S U M M A R Y
DOSAGE = 0.85 MG/L  USAGE = 16.1 KG  SCALE READING = 46.1 KG
                                     MAX READING   = 55.2 KG
   RESIDUAL = 0.00 MG/L              MIN READING   = 35.8 KG
```

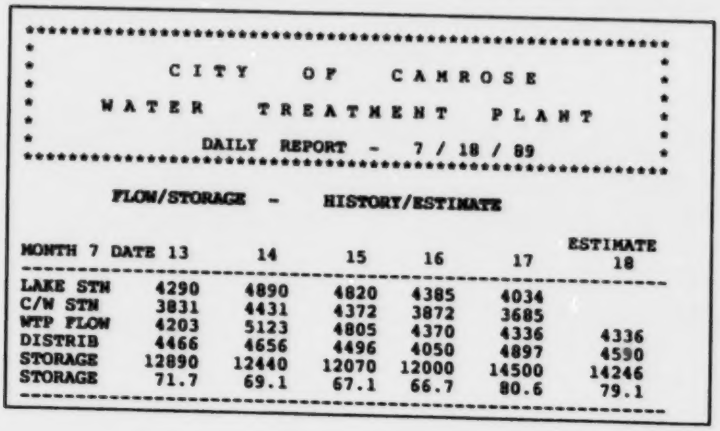
3) **Four Hour Summary Report - (Figure 5)** - Every four hours a report is printed which summarizes current flows, turbidities and chemical residuals.

4) **Backwash Report - (Figure 6)** - At the completion of each backwash, a report is printed which summarizes backwash data such as:

- which filter backwashed
- filter parameters at time of backwash
- reason for filter backwash
- how much water was used backwashing the filter
- filter production
- filter run time

Figure 4

Water Treatment Plant - Consumption/History Report



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*************************************************************
*                                                           *
*          C I T Y     O F     C A M R O S E                *
*                                                           *
*    W A T E R     T R E A T M E N T     P L A N T          *
*                                                           *
*              DAILY  REPORT  -   7 / 18 / 89               *
*************************************************************

        FLOW/STORAGE   -    HISTORY/ESTIMATE

                                                     ESTIMATE
MONTH 7 DATE 13        14         15        16         17          18
--------------------------------------------------------------
LAKE STN     4290      4890       4820      4385       4034
C/W STN      3831      4431       4372      3872       3685
WTP FLOW     4203      5123       4805      4370       4336        4336
DISTRIB      4466      4656       4496      4050       4897        4590
STORAGE     12890     12440      12070     12000      14500       14246
STORAGE      71.7      69.1       67.1      66.7       80.6        79.1
--------------------------------------------------------------
```
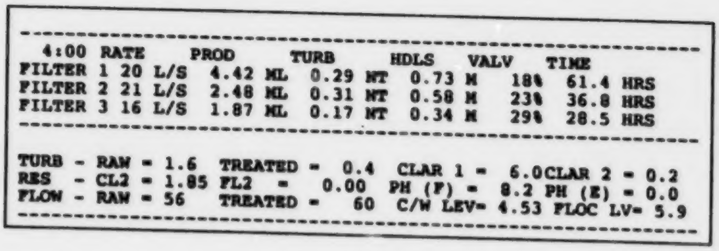

FIGURE 5

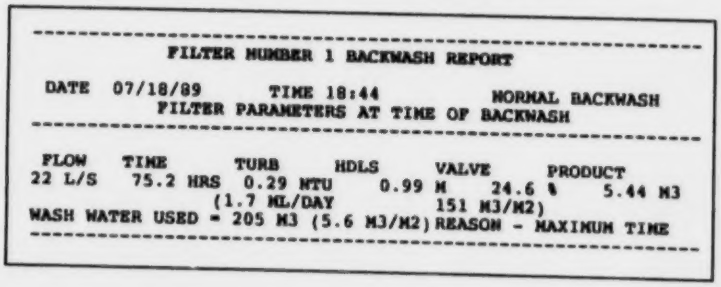
Water Treatment Plant - Four Hour Status Summary



```
----------------------------------------------------------------
  4:00 RATE      PROD      TURB      HDLS    VALV    TIME
FILTER 1 20 L/S  4.42 ML  0.29 NT  0.73 M    18%  61.4 HRS
FILTER 2 21 L/S  2.48 ML  0.31 NT  0.58 M    23%  36.8 HRS
FILTER 3 16 L/S  1.87 ML  0.17 NT  0.34 M    29%  28.5 HRS
----------------------------------------------------------------

TURB - RAW = 1.6  TREATED =  0.4  CLAR 1 =  6.0CLAR 2 = 0.2
RES  - CL2 = 1.85 FL2   =   0.00  PH (F) =  8.2 PH (E) = 0.0
FLOW - RAW = 56   TREATED =    60  C/W LEV= 4.53 FLOC LV= 5.9
----------------------------------------------------------------
```

FIGURE 6

Water Treatment Plant - Backwash Report



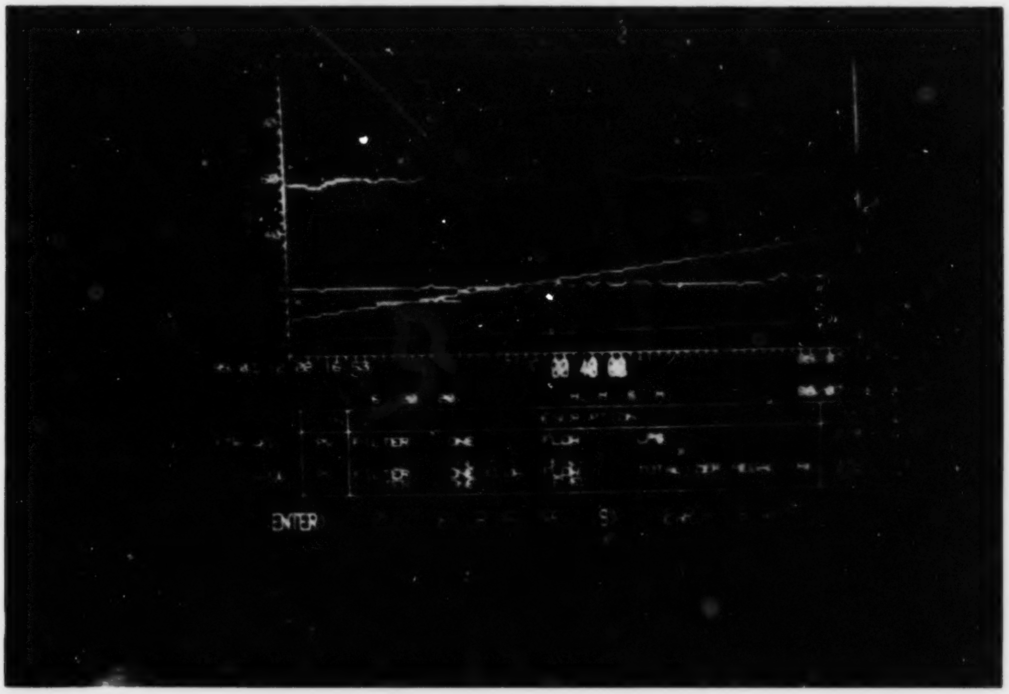
```
--------------------------------------------------------------
              FILTER NUMBER 1 BACKWASH REPORT

 DATE  07/18/89       TIME 18:44           NORMAL BACKWASH
             FILTER PARAMETERS AT TIME OF BACKWASH
--------------------------------------------------------------

 FLOW   TIME        TURB      HDLS      VALVE      PRODUCT
22 L/S   75.2 HRS  0.29 NTU     0.99 M    24.6 %     5.44 M3
                  (1.7 ML/DAY          151 M3/M2)
WASH WATER USED = 205 M3 (5.6 M3/M2)REASON - MAXIMUM TIME
--------------------------------------------------------------
```

PRINTER TWO

5) **Status/Alarm Summary** - Printer Two is dedicated to printing a hard copy of alarm and equipment status changes. This report prints the time and date of any alarm or equipment change of status.

6) **Trending Graphs** – (**Picture 2**) – The monitoring system at the Water Treatment Plant allows the operator to select any process parameter, and it will produce a graph of that parameter with respect to time. The system also allows the operator to print the graph when required.

PICTURE 2
Trending Graph Of Filter Operation



7) **Event Archiving** – The monitoring system also automatically archives all operator key strokes so that they can be reviewed or printed via Printer Two when required.

PHONE IN REPORT – (Figure 7)

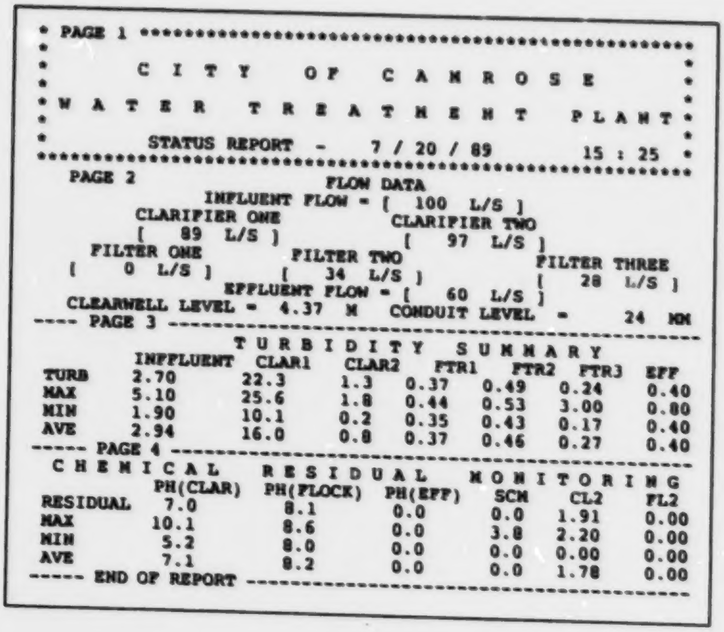
A phone-in report is available to the operators from either a portable computer which is available for their use when on call, or from any home computer equipped with a modem. The report summarizes all flows, level, turbidities, and chemical parameters measured in the Plant. Future modifications may include alarm information and some remote control features.

With the Water Treatment Plant now fully programmed, a large segment of the tedious work which would normally have to be done by the operators is now accomplished by the PLC and monitoring system. The Plant operates in a completely automatic mode 24 hours per day, and is manned only eight hours per day on weekdays, and 3 hours per day on weekends.

If a critical alarm occurs when the Plant is not manned, the alarm system pages, an "on-call" operator. If the operator does not respond to the page, the system then phones the operator at home. If there is still no response from the operator, the system calls other operators until a response is finally obtained.

FIGURE 7

Water Treatment Plant - *Phone In Report*



```
* PAGE 1 ***************************************************
*                                                          *
*       C  I  T  Y    O F   C  A  M  R  O  S  E            *
*                                                          *
*W  A  T  E  R    T  R  E  A  T  M  E  N  T    P  L  A  N  T*
*                                                          *
*        STATUS REPORT  -    7 / 20 / 89          15 : 25  *
************************************************************
  PAGE 2                     FLOW DATA
                INFLUENT FLOW = [  100  L/S ]
        CLARIFIER ONE                 CLARIFIER TWO
        [   89  L/S ]                 [   97  L/S ]
     FILTER ONE           FILTER TWO                  FILTER THREE
  [     0  L/S ]          [   34  L/S ]               [    28  L/S ]
                   EFFLUENT FLOW = [   60  L/S ]
  CLEARWELL LEVEL =  4.37  M   CONDUIT LEVEL  =      24  MM
---- PAGE 3 ------------------------------------------------
                   T U R B I D I T Y   S U M M A R Y
          INFFLUENT  CLAR1    CLAR2   FTR1    FTR2   FTR3   EFF
TURB      2.70       22.3     1.3    0.37    0.49    0.24    0.40
MAX       5.10       25.6     1.8    0.44    0.53    3.00    0.80
MIN       1.90       10.1     0.2    0.35    0.43    0.17    0.40
AVE       2.94       16.0     0.8    0.37    0.46    0.27    0.40
----- PAGE 4 -----------------------------------------------
 C H E M I C A L    R E S I D U A L    M O N I T O R I N G
            PH(CLAR)  PH(FLOCK)  PH(EFF)   SCM     CL2     FL2
RESIDUAL     7.0        8.1       0.0      0.0     1.91    0.00
MAX         10.1        8.6       0.0      3.8     2.20    0.00
MIN          5.2        8.0       0.0      0.0     0.00    0.00
AVE          7.1        8.2       0.0      0.0     1.78    0.00
----- END OF REPORT ----------------------------------------
```

The control system at the Camrose Water Treatment Plant has now been essentially complete for approximately one and on half years, and has proven to be a reliable and efficient system.

CONCLUSION

Computer control of water or wastewater pumping or treatment systems can be implemented at a reasonable cost, and, if enough emphasis is placed on the programming and the operator interface, can be user friendly, this can relieve the operator from any tedious tasks.

PLC systems are simply to install, and simple to maintain. If properly programmed with adequate documentation, operators will easily be able to trouble shoot most problems without requiring expertise in PLC programming or instrumentation.

The experience gained with the City of Camrose PLC/Computer Monitoring Systems over the last six years can be summarized in the following recommendations aimed at Consulting Engineers or any potential user:-

1] Input Output Wiring

Ensure that ample room is left for future expansion. Extra wires should be pulled in each conduit, and ample space should be available in the PLC cabinet. In the Camrose Water Treatment Plant, each conduit was specified to contain 30% spare wires. In many locations in the Plant, this did not turn out to be enough and extra conduit had to be installed after the initial commissioning.

PICTURE 9
Water Treatment Plant - Remote Terminal For Filter Control and Monitoring

Emphasis should be placed on the preparation of electrical/instrumentation drawings showing connection of process systems to the PLC rack. These should not be in schematic format but should be full wiring diagrams showing wire numbering, panel layout and fuse and terminal strip locations.

Ensure that adequate electrical/instrumentation inspection is maintained throughout construction so that any problems are identified early.

Ensure accurate as-built drawings are prepared.

2) PLC Programming

The plant operator must be involved in the programming process.

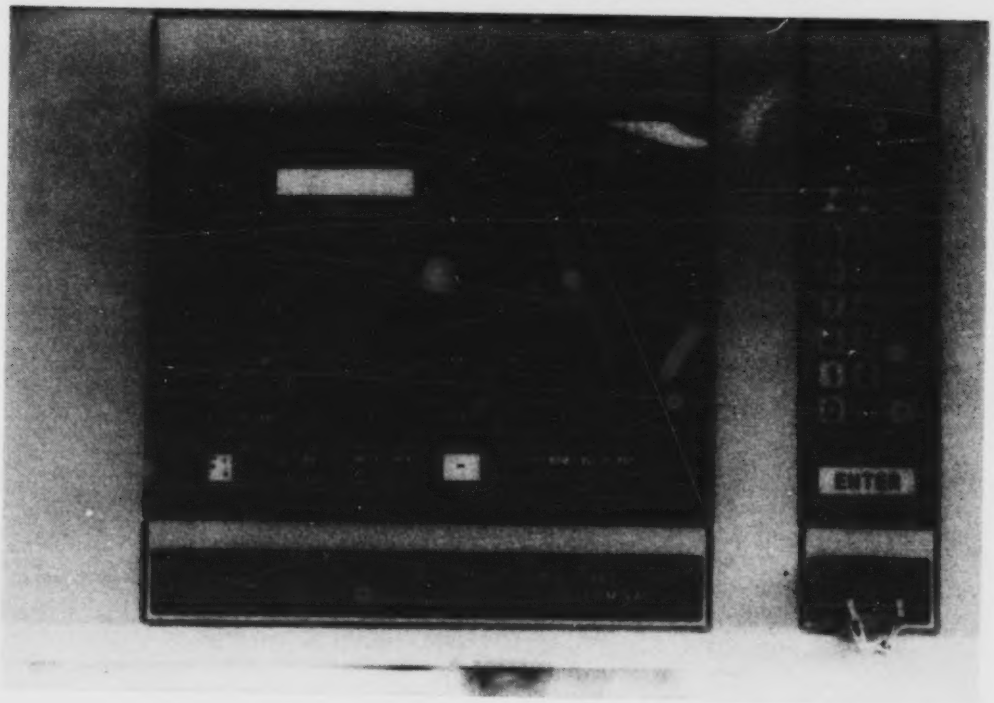
The PLC program should be installed in stages if possible so that enhancements can be tailored to the particular process, and operator requirements as experience is gained with each particular process.

Complete program documentation, in English, is essential if the operators are to be able to trouble shoot operational problems.

Each alarm must have a complete description of the factors which actuate it, and how it may be reset.
Each process must have a complete description of its operation.

PICTURE 4

Water Treatment Plant - Filter Status Displayed on Remote Terminal



3) Operator Interface - Monitoring/Control System

The Operator Interface, or Monitoring/Control is the most important part of the system. If not properly programmed, the entire process will be difficult to control, awkward to use, and will create unnecessary work for the operator.

The operator must have input in the development of screens and control/monitoring features.

Extra effort should be made to ensure that the system is as user friendly as possible.

Design the system to give the operator as much control of the system setpoints, and operations as possible.

The computing power of the PLC should be utilized to generate useful reports for the operator. There is virtually no limit to the amount of information which can be calculated and reported by the PLC system.
It is essential that the Design Engineer work closely with the Electrical Consultant, the Programmer and the Operator in order to produce a system which is user friendly and which meets all of the operator's requirements.

It is hoped that some of these recommendations will be of use to Consulting Engineers, and future owners of computer controlled stations.

PICTURE 5

Water Treatment Plant Main Control Station

PICTURE 6

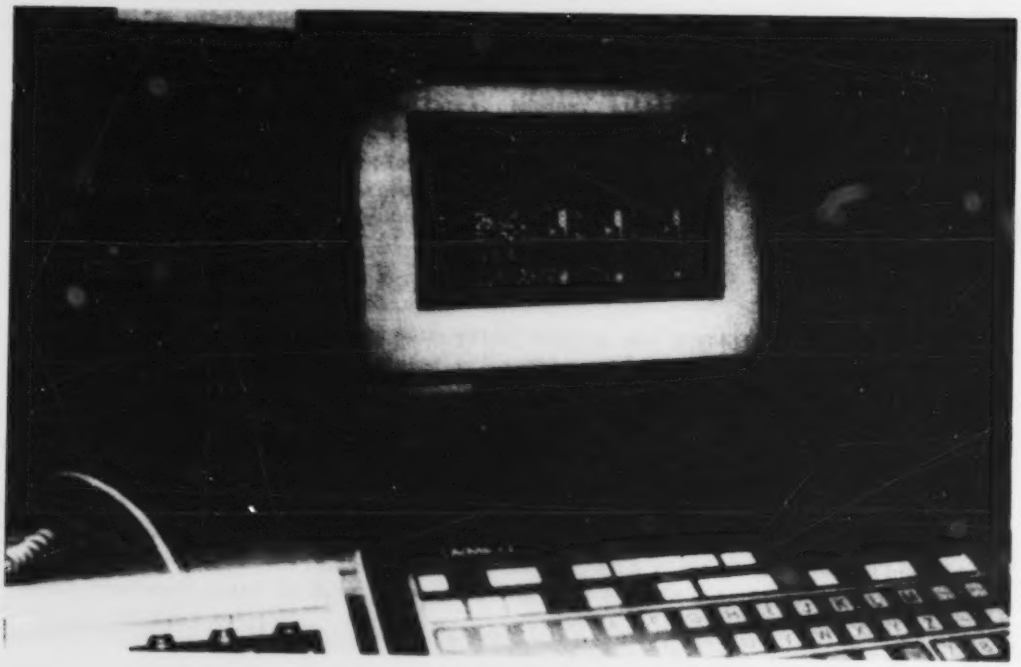
Water Treatment Plant - Main Control Display - Clarifier Operation

PICTURE 7

Water Treatment Plant - Main Control Display - Filter Status

PICTURE 8

Water Treatment Plant - Main Control Display - Filter Setpoints



HENRY KROEGER REGIONAL WATER SERVICES COMMISSION
ALBERTA REGIONAL UTILITIES PROGRAM

Peter Brouwer
Alberta Transportation and Utilities

INTRODUCTION

The growth of urban & rural areas in Alberta in the 1970's combined with the increase of demand for municipal services and the province's philosophy that financial and environmental benefits could be obtained by evaluating servicing on a regional basis led the province into forming the Regional Utilities Program. The program provides financial and technical assistance to groups of municipalities for the construction of water supply or sewage treatment projects which involve several municipalities utilizing common supply and treatment facilities. This paper will provide a brief overview of the most recently completed system, the Henry Kroeger Regional Water Services Commission pipeline.

THE PROBLEM

The Special Areas of Alberta (figure 1) is located in a semi arid area of the province with minimum average rainfall of 12 inches and high evapotranspiration rates.(1) The area is plagued with poor quantity of quality of water. Since the early 1970's numerous independent studies where under taken by the municipalities of Hanna, Oyen, Cereal and Youngstown in order to locate a suitable source and quality of potable water. All of the municipalities involved relied on groundwater as their source of supply except Hanna who relied on surface run off into two shallow reservoirs, namely the CNR and Helmer.

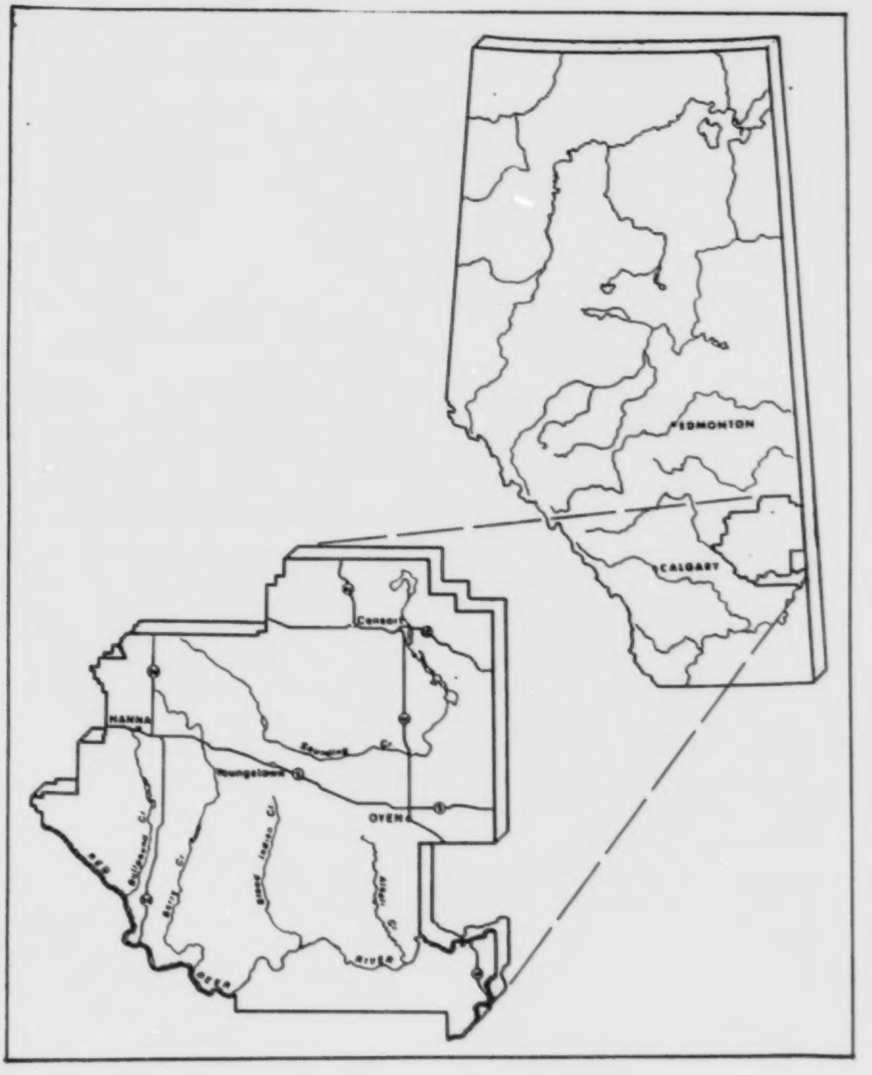
Groundwater supplies were located in close proximity to the municipality, however was plagued with high total dissolved solids, sulfates, sodium, alkalinity and calcium and magnesium hardness (2). Besides poor quality, both Youngstown and Oyen had problems meeting peak summer demands which often led to over pumping of wells. In early 1984 the situation had become so critical that the Town of Oyen was forced to haul water from Acadia Valley, 35 km away to meet their needs.

HISTORY

As a result of obvious shortages in both water quality and quantity, the then Department of Utilities and Telecommunications initiated a planning study to assess and evaluate alternative sources of water supply to serve the communities of Oyen, Youngstown, Cereal, Acadia Valley and several hamlets in between. The study consultant was responsible to a steering committee comprised of representatives from each of the municipalities involved and the department. The study evaluated several alternatives ranging from individual groundwater systems to a regional pipeline source from the Red Deer River. The study recommended that a water supply line be constructed from the Red Deer River to Acadia Valley, Oyen, and on to Youngstown at an estimated cost of $16 million. (2) The alternative included a batch plant in Acadia Valley and a full scale water treatment plant in Oyen. The pipeline from the Red Deer River was sized to provide for municipal purposes only.

The consultant recommended that the steering committee develop a working paper to identify the benefits, costs and the concept of upgrading the recommended alternative to include capacity for irrigation purposes. The steering committee agreed with this recommendation since the concept of irrigation in the "Special Areas" had been actively promoted among local residents for years.

FIGURE 1
Special Areas Location Plan


EDMONTON
CALGARY
HANNA
Consort
Youngstown
OYEN
Sounding Cr
Bullpound Cr
Berry Cr
Blood Indian Cr
Alkali Cr
RED DEER RIVER

The steering committee approached the Department of Environment, who commissioned a study which would assess the economic viability of combining the municipal supply with an irrigation component.

The study consultant provided the following conclusions:

1) That the combination of a municipal/irrigation water supply project is not economically viable.

2) That the steering committee proceed to implement the construction of a treated water pipeline from Hanna to Oyen stopping in Oyen at an estimated project cost of $11 million. It was concluded that Acadia Valley had an ample supply of suitable quality water for municipal purposes for the next 25 years. (3)

The steering committee indicated that water supply for irrigation and domestic purpose were important to the economic viability of the area. However, the committee agreed to proceed with the municipal portion now, as long as the irrigation proceeded in the future.

The steering committee proceeded to lobby both levels of government for a funding commitment for the implementation of the project. On October 13th, 1988 the Minister of Transportation and Utilities approved the project for a special 75% grant to a maximum project cost of $11 million.

The funding approval was subject to:

1) A regional commission being formed consisting of member municipalities for the construction, administration, ownership and operation of the regional facility.

2) Confirmation of the financial and legal arrangements regarding the supply of water from the Town of Hanna.

Hanna's facilities consist of a recently completed water treatment located within the bounderies of town with enough capacity to serve a population of 3900. Since present population of Hanna is 3036, enough capacity was available to connect Oyen, Youngstown and Cereal.

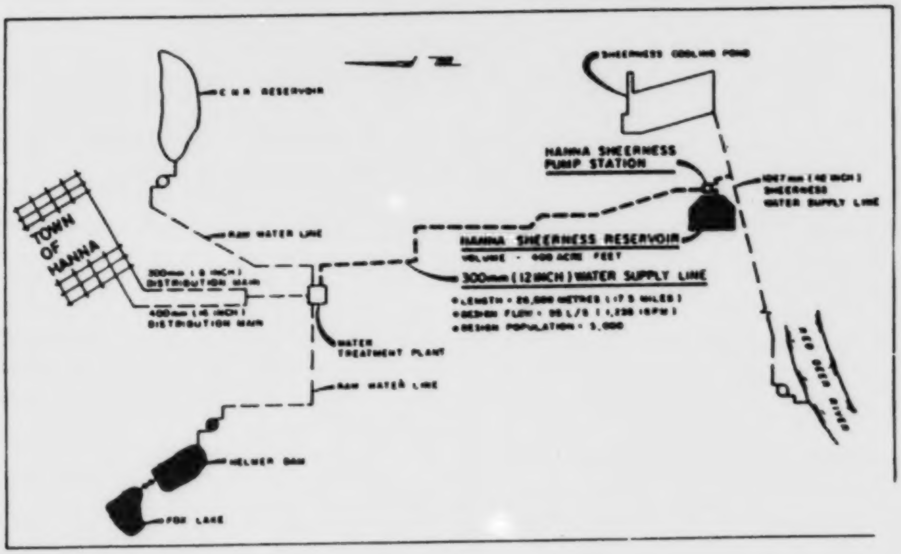
Hanna obtains its raw water through a pipeline from the Sheerness Power Plant located 28 km south of town. The Sheerness Power Plant obtains its water for cooling from (figure 2) a series of pumps located at the Red Deer River and a 42" pipeline. When the pumpstation and pipeline was designed, Alberta Environment purchased 26 C.F.S. of capacity in anticipation of serving the Town of Hanna and some irrigation needs. (4)

The Federal Government through the Prairie Farm Rehabilitation Administration also committed an additional 15 percent to the project costs. The local share was to be obtained through a 25 year debenture repayment through the Alberta Mortgage and Finance Corporation which would be repaid through the water rates set by the commission.

On February 11, 1988 the "Henry Kroeger Water Services Commission" was officially formed by an Order in Council under the Regional Municipal Services Act. The Commission consists of five member municipalities that is: Hanna, Youngstown, Cereal, Oyen and the Special Areas Board. Under the act the Commission has the same powers and rights as any other municipality within the province. It should also be noted that under the act the Commission must operate on a break even scenario and must charge the same water rate to each of the participating municipalities of the commission.

FIGURE 2

Sheerness Power Plant Pump Locations


C N R RESERVOIR
SHEERNESS COOLING POND
HANNA SHEERNESS PUMP STATION
1067mm (42 INCH) SHEERNESS WATER SUPPLY LINE
RAW WATER LINE
TOWN OF HANNA
200mm (8 INCH) DISTRIBUTION MAIN
400mm (16 INCH) DISTRIBUTION MAIN
HANNA SHEERNESS RESERVOIR
VOLUME - 400 ACRE FEET
300mm (12 INCH) WATER SUPPLY LINE
•LENGTH = 28,000 METRES (17.5 MILES)
•DESIGN FLOW = 95 L/S (1,238 IGPM)
•DESIGN POPULATION = 3,000
WATER TREATMENT PLANT
RAW WATER LINE
HELMER DAM
FOX LAKE
RED DEER RIVER

IMPLEMENTATION

As funding had now been allocated for the project, the Commission immediately retained the services of a consultant to prepare a pre-design report. The work involved preparing a functional design for the construction of a treated water line between Hanna and Oyen.

Although average annucl water consumption of 100 igpcd and maximum day peaking factors of 2.5 would provide a basic level of service, closer evaluation of historical water consumption and peaking indicated that these criteria would not suffice. As a result, average annual consumptions of 110 igpcd with peaking factors of 3.0 were chosen as basic design parameters. [5] The pipeline was designed to meet maximum day flows, with each member municipality responsible for providing suitable on-site storage and distribution to meet maximum hour requirements.

Four types of pipe material were investigated in the pre-design stage. Steel pipe with polyethylene coating and cathodic protection, polyethylene pipe, Series PVC pipe and Class PVC pipe were reviewed. Preliminary cost estimates concluded that both PVC class pipe and polyethylene were not cost comparative at diameters required, and further, exceeded the upset project budget of $11 million.

Further analysis concluded that steel pipe had substantially higher operating costs than PVC series, since the steel pipe was only economical in smaller diameters, which resulted in increased head loss (ie. higher operating costs). As a result, the recommended pipe material was Series PVC, based on the assumption that the pipeline be designed to withstand 90 percent of pressure rating (series number) equal or exceeding anticipated maximum static pressure and 80 percent of pressure rating equal or exceeding normal operating pressures under pumping condition.

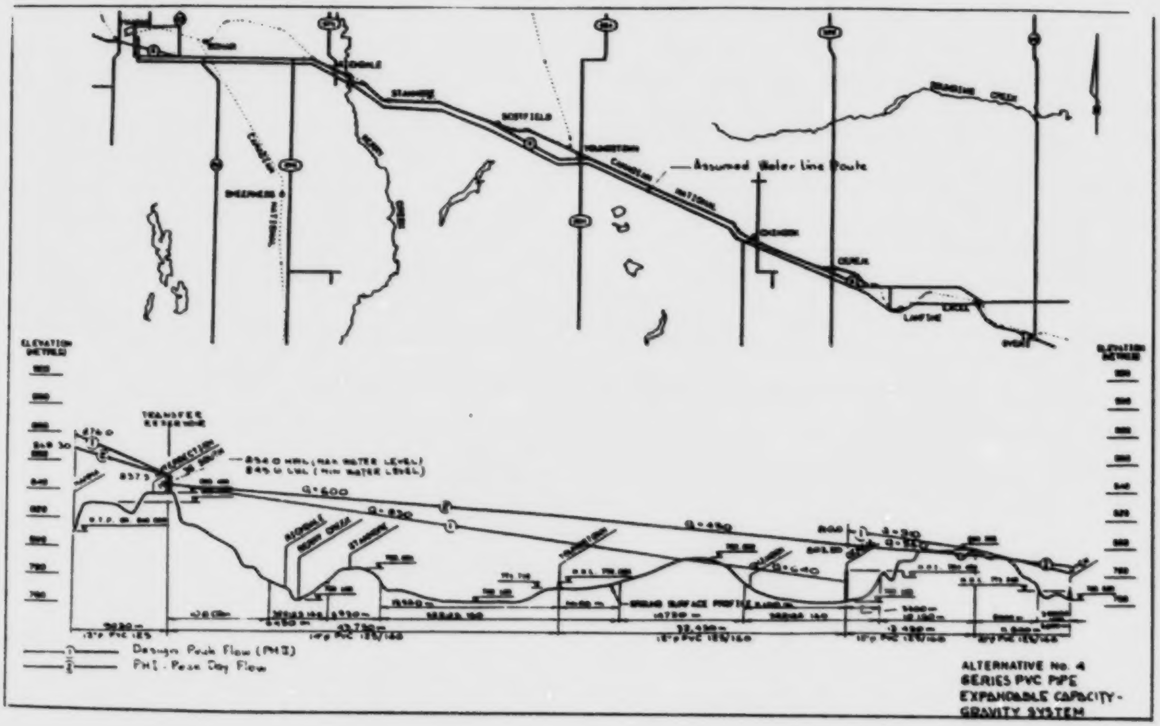
As part of the pre-design work the consultant was required to prepare a functional design of the proposed system. A number of concepts were reviewed and subsequently reduced to two alternatives, which were analyzed in greater detail.

The first alternative proposed the upgrading of pumps at the Hanna water treatment plant so as to pump treated water to an elevated storage reservoir at the height of land at the intersection of Highway 9 and 36 south. From there the elevated reservoir would serve the municipalities of Youngstown, Cereal, Oyen, 5 Hamlets and rural connections by gravity. (figure 3)

FIGURE 3

Alternative No. 4

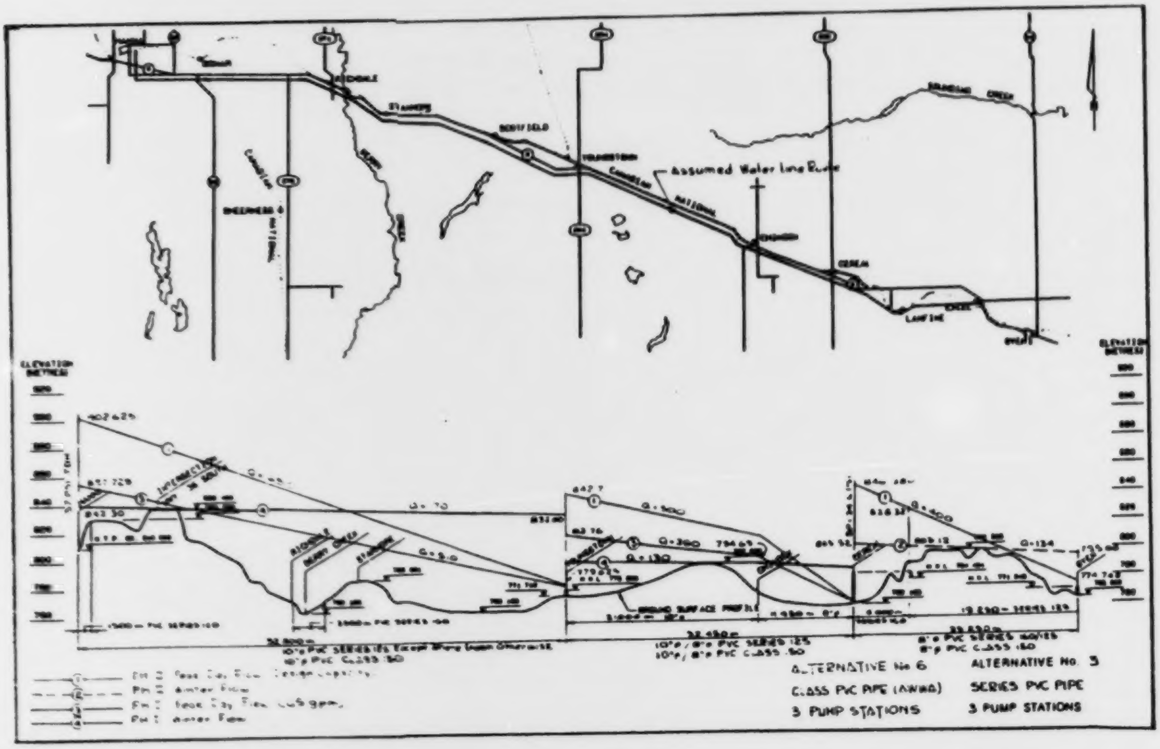
Series PVC Pipe Expandable Capacity-Gravity System


Assumed Water Line Route
ELEVATION (METRES)
TRANSFER RESERVOIR
854.0 HWL (MAX WATER LEVEL)
845.0 LWL (MIN WATER LEVEL)
Q=600
Q=830
Q=450
Design Peak Flow (PHD)
PHD - Peak Day Flow
ALTERNATIVE No. 4
SERIES PVC PIPE
EXPANDABLE CAPACITY-
GRAVITY SYSTEM

The second alternative would eliminate the need for an elevated reservoir, however, would require upgrading of pumps within the Hanna water treatment plant and additional pump stations at Youngstown and Cereal. (figure 4)

In order to determine which alternative would be more advantageous, the advantages and disadvantages of both systems were listed.

FIGURE 4
Alternative No. 6 - Class PVC Pipe (AWWA) 3 Pump Stations
Alternative No. 3 - Series PVC Pipe 3 Pump Stations


Assumed Water Line Route
ELEVATION (METRES)
GROUND SURFACE PROFILE
ALTERNATIVE No 6
CLASS PVC PIPE (AWWA)
3 PUMP STATIONS
ALTERNATIVE No. 3
SERIES PVC PIPE
3 PUMP STATIONS

ELEVATED RESERVOIR & GRAVITY SYSTEM

ADVANTAGES

- Simple operation, minimal controls
- Lower pumping costs
- Future capacity for booster stations
- Reduce likelihood of air entrance due to power failures

DISADVANTAGES

- Cost of construction of a 275,000 gallon balancing reservoir
- Reservoir maintenance costs

CONTINUOUSLY PUMPED SYSTEM

ADVANTAGES	**DISADVANTAGES**
- Elimination of a reservoir	- Reduction in system reliability
	- Increase need for pump controls (SCADA)

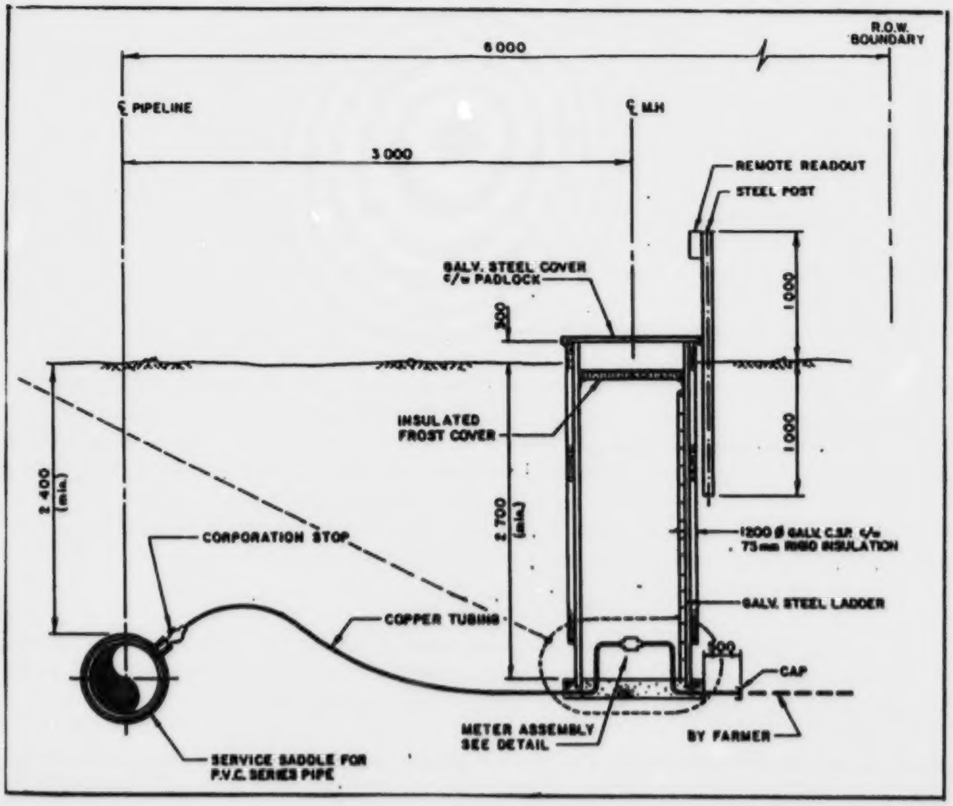
Considering the above breakdown of advantages versus disadvantages it was recommended to the Commission that the reservoir and gravity concept be selected for implementation.

RURAL CONNECTIONS

Several rural landowners situated along Highway 9 expressed interest in connecting into the main supply line between Hanna and Oyen. Due to the high capital costs involved in the construction of the entire system (from the Red Deer River to Oyen) and the finite number of gallons of water available per day through the pipeline, the commission decided to allow average day consumptions of 1/2 igpm or 720 igpd for household use for each rural connection. The connection consists of a taping saddle similar to a municipal corporation stop. Piping connects to a manhole within the Right of Way which contains a double check valve, flow meter, a flow control valve and if necessary a pressure reducing valve (figure 5).

FIGURE 5

Farm Service Connection Detail


R.O.W. BOUNDARY
6000
℄ PIPELINE
℄ M.H
3000
REMOTE READOUT
STEEL POST
GALV. STEEL COVER c/w PADLOCK
300
1000
INSULATED FROST COVER
1000
2400 (min.)
2700 (min.)
CORPORATION STOP
1200 Ø GALV. C.S.P. c/w 75mm RIGID INSULATION
GALV. STEEL LADDER
COPPER TUBING
300
CAP
METER ASSEMBLY SEE DETAIL
BY FARMER
SERVICE SADDLE FOR P.V.C. SERIES PIPE

The pipe is stubbed outside the manhole which is the tie-in point for the rural user. As the daily flow is minimal, 1000 gallon cisterns located at each rural users home acts as suitable storage for distribution.

AIR RELIEF VALVES

Contrary to popular belief, the prairies of south eastern Alberta are not flat as a dried up lake bottom. In fact the quite hilly terrain, especially between Cereal and Oyen, necessitated the installation of over 150 air relief valves. Standard design procedures would have placed an automatic air/vacuum relief valve enclosed within a concrete or CSP manhole. At an estimated cost of approximately $3000 each, this would have added an additional $450,000 to the project costs and more importantly would increase the annual maintenance costs of both the manhole and the actual valve, as problems with freezing manholes are common where electrical power is not available.

Based on the preceding the idea of installing manual air relief valves [figure 6] were evaluated and were recommended for the following reasons:

1] The gravity reservoir concept would reduce the chances of air entertainment therefore there would be less need for air expulsion.

2] The capital costs are significantly lower.

3] The reduction in operation and maintenance costs.

LAND ACQUISITION

By the end of April 1988 the Commission had approved the gravity concept as the most favorable alternative. Several alternatives were presented to the Commission regarding the route alignment, however of these, three alternatives were given serious consideration. Those being the pipeline located:

1] within the south Highway Right of Way

2] between the north Highway Right of Way and the Railway tracks

3] 30m south of the Highway Right of Way

Cost/Benefit analyses conducted, concluded that there would not be any advantage in constructing within either the Highway or Railway Right of Way as the increase in construction costs would be far in excess of the payments necessary to local residents for right of entry, crop damages and inconvenience costs for structures such as manholes and air relief valves. The Commission agreed to construct the pipeline outside both the railway and highway right of way. The Department of Transportation and Utilities stipulated the need for a 30m buffer between the edge of their right of way and the centre of the pipeline in case the highway is ever twined in the future.

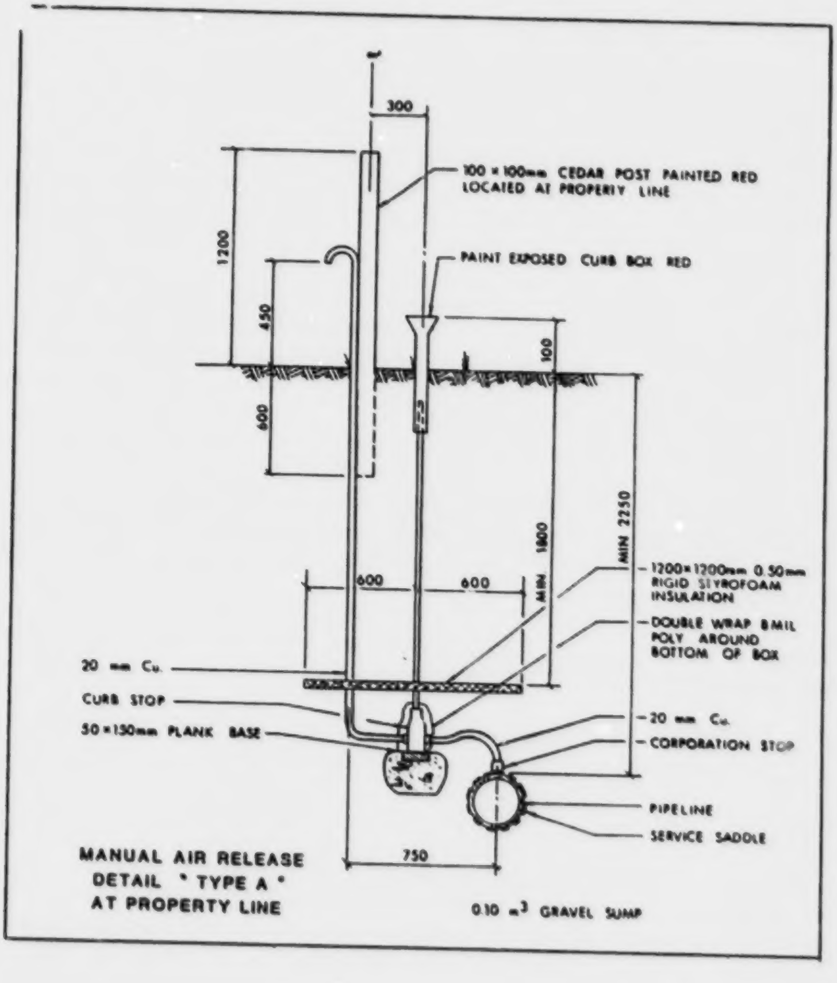
Upon defining the alignment, the Commission retained the services of an independent land consultant to acquire the necessary lands. In order to instill a sense of fair treatment amongst all land owners a "Town Hall meeting" concept was implemented.

Each affected landowner was personally invited to attend a meeting held in the local area where

Commission representatives provided the terms of compensation available for right of entry, crop losses/damages and structure installation.

This process allowed the local landowners opportunity to ask questions and more importantly assured himself that he was being treated the same as his neighbor. Acquisition commenced on May 1/88 and by May 20/88; 90 percent of the necessary lands were acquired, with the remaining lands acquired by mid June of 1988.

FIGURE 8
Manual Air Release Detail


300
100 × 100mm CEDAR POST PAINTED RED LOCATED AT PROPERTY LINE
1200
PAINT EXPOSED CURB BOX RED
450
100
600
MIN 2250
MIN 1800
600
600
1200×1200mm 0.50mm RIGID STYROFOAM INSULATION
DOUBLE WRAP 8MIL POLY AROUND BOTTOM OF BOX
20 mm Cu.
CURB STOP
50×150mm PLANK BASE
20 mm Cu.
CORPORATION STOP
PIPELINE
SERVICE SADDLE
750
MANUAL AIR RELEASE DETAIL " TYPE A " AT PROPERTY LINE
0.10 m³ GRAVEL SUMP

RECLAMATION

In recent years, there has been a larger emphasis placed on reclamation of lands affected as a result of pipeline construction. At the request of the Land Reclamation Council a pedology study was undertaken by the Prairie Farm Rehabilitation Administration in order to determine the most appropriate stripping and topsoil replacements methods for the area. Approximately 160 boreholes 8 feet deep were drilled along the 110 kilometre pipeline route.

As a result of these soil analyses the PFRA recommended four methods of stripping along different sections of the pipeline. The methods ranged from no stripping (ie. mixing topsoil and subsoil) to full 30 meter Right of Way stripping. These methods were then incorporated with the appropriate contract documents.

TENDERING AND CONSTRUCTION

Due to the sheer length of the pipeline and the narrow window of construction available, it was decided to split the pipeline design and construction into three contracts.

One contract between Hanna and Youngstown, another between Youngstown and Cereal and the final contract between Cereal and Oyen. This provided the opportunity of receiving more competitive bids for both supply and installation of the pipe as tenders closed on successive days for a three day period.

In addition, to the three pipeline contracts two others tender packages were prepared, one for modifications at the water treatment plant involving additional pump installations, computer logic upgrading and upsizing of the raw water supply pump at Sheerness in order to supply sufficient quantities of raw water to the treatment plant. The second tender was let for the construction of a 275,000 gallon reservoir located at the height of land, located at the intersection of highway 9 and 36 south. This tender was let as a design build contract.

Pipeline construction commenced July 1988 by three contractors and by September the mainline had been virtually installed. One contractor was installing approximately 850 meters of 14" per day per crew during peak production. The Special Areas Board allowed open cutting of gravel roads which made pipe installation simpler.

Construction proceeded on the elevated storage reservoir and the pump and computer control upgrading during the months of October and November.

During the months of October and November the mainline was continually flushed and pressure tested and on December 5th, 1988 the system was considered substantially complete.

CONCLUSION

The lack of a good source of quality water for residents of the Special Areas has been a problem since it was first settled in the early 1920's. So much so, that economic growth of the area has lagged behind the rest of the province. These municipalities have implemented the necessary infrastructure required for steady growth and even the potential of attracting industries to the area.

With the success of the regional system, the communities of Delia and Craigmyle are now considering connecting to the regional pipeline. Also, regional systems, such as the proposed Highway 14 waterline from Sherwood Park to Viking are now being planned for implementation.

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PLATE SETTLERS IN WATER TREATMENT

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INTRODUCTION

Gravity settling, or sedimentation, is a process in which the force of gravity acts on particles and causes them to settle. The suspended particles are heavier than water and, provided the flow through the settling tank is lower than the settling velocities of the particles, the particles will undergo various settling phenomena. The sedimentation process is widely used in water and wastewater treatment.

Horizontal flow settling tanks are the most common sedimentation tanks used in water treatment. Because of their simplicity, they are easy to build, operate and maintain. Horizontal settling tanks do however, require a relatively large surface area. The concept of introducing plate settlers in settling tanks increase the flow per unit plan area.

Although the theoretical possibilities of plate settlers were discussed as early as the late 1800's, it was not until the late 1970's that the first full-scale application of plate settlers was observed in Amsterdam, Netherlands (136 ML/d).

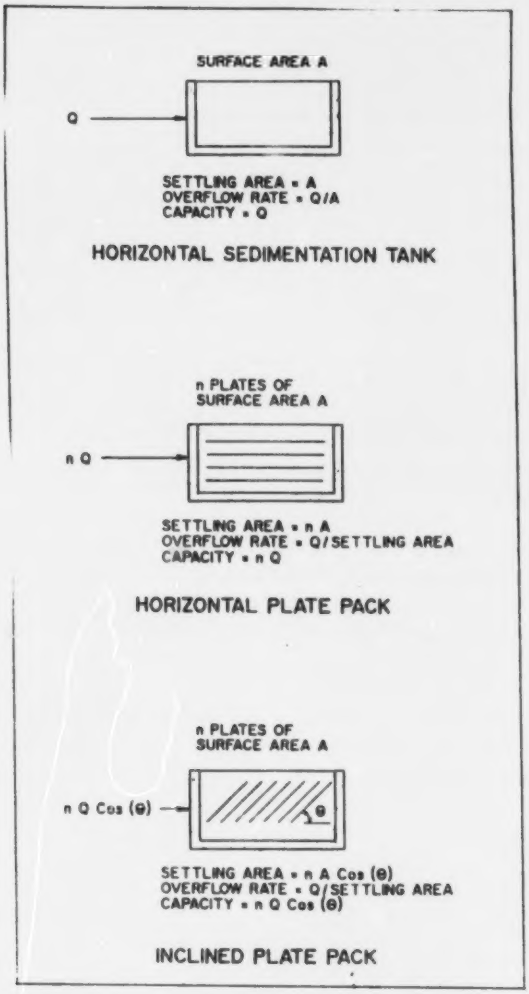
Since then, units have been installed in Germany, France, Canada, United States and other countries.

Theory

The theory behind the use of plate settlers in the sedimentation process is based on the fact that particle removal in a settling tank is a function of horizontal surface area, i.e. the amount of surface area available for the particle to settle on. Conventional settling basins require a certain depth to prevent scouring of the settled solids and a sufficient detention time to allow an acceptable fraction of the particles to settle on the bottom of the basin. Looking simply at the theory of plate settlers, it can be compared to inserting 'n' horizontal plates in a settling tank, and thereby having 'n' shallow-depth settling tanks, each with a settling area 'A', see Figure 1 below. Therefore, at the same overflow rate per horizontal plate, a tank with 'n' horizontal plates will have 'n' times the capacity of a tank without plates.

In practice, the plates in plate settlers are inclined at an angle of 55 to 60 degrees to allow the sludge to be removed by gravity. This reduces the capacity of the plate settler arrangement somewhat, depending on the angle of inclination of the plates, but eliminates an otherwise difficult sludge removal problem. The settling area of a tank containing plate settlers is based on the horizontally projected area, or ['A' x cosine of the angle inclination].

FIGURE 1


SURFACE AREA A
Q
SETTLING AREA = A
OVERFLOW RATE = Q/A
CAPACITY = Q
HORIZONTAL SEDIMENTATION TANK
n PLATES OF SURFACE AREA A
n Q
SETTLING AREA = n A
OVERFLOW RATE = Q/SETTLING AREA
CAPACITY = n Q
HORIZONTAL PLATE PACK
n PLATES OF SURFACE AREA A
n Q Cos (θ)
θ
SETTLING AREA = n A Cos (θ)
OVERFLOW RATE = Q/SETTLING AREA
CAPACITY = n Q Cos (θ)
INCLINED PLATE PACK

For ease of calculation, assume the angle of inclination is 60 degrees. The cosine of 60 degrees is 0.5 therefore, a tank containing 'n' plates inclined at 60 degrees would have a total settling area of 'n' x 'A' x 0.5. To assist in gaining an appreciation for the effect of plates, assume that a tank contains 10 plates each with an area 'A'. Such a tank with plates, would have a capacity of (10 x 0.5) or 5.0 times the capacity of a plain settling tank of the same area 'A'. Obviously, the tank with plates will require much less surface space or plan area than a similar capacity settling tank without plates.

Plate Settler Performance

Some of the variables which affect the performance of plate settlers are:

- plate length
- plate angle
- plate spacing

- length to width ratio
- flow direction
- influent and effluent configurations
- overflow rate
- solids loading

Several of the above variables have been tested extensively, and the experimentally determined results have become accepted. For instance, the plate spacing is usually 50 millimeters. If the plate settlers are operated in a counter current mode, the plate angle is 55 to 60 degrees to allow for automatic sludge removal. Other parameters, such as overflow rate, are more variable depending on the nature of solids in the water and on the solids loading.

Solids generated in a colour removal plant behave differently than solids in a turbidity removal plant. The influence of lime or limestone in Water Treatment plants ahead of settling will also effect the selection of overflow rate.

Pilot testing of plate settlers in water treatment plants has been conducted by Gore & Storrie at two locations in Ontario. One plant is a turbidity removal process, while the other is a colour removal process. The two flocs generated have different characteristics and therefore, different plate settlers design requirements. Following is a brief summary of the tests conducted at each location.

Pilot Testing at a Colour Removal Water Treatment Plant

The raw water at this plant is a soft, coloured water, with an average colour of approximately 30 Hazen Colour Units (HCU). A floc is generated by adding alum and activated silica to the water and passing the water through three flocculation chambers. In the existing plant, the flocculated water then enters a rectangular shaped settling tank. The colour floc generated at this plant is large and fluffy, but very fragile and light.

The pilot testing program was conducted to assess the performance of plate settlers relative to conventional settling tanks. The efficiencies of the plate settlers and the conventional tanks were evaluated by measuring the solids removal obtained in both; i.e. influent and effluent suspended solids measurements were taken and a percent removal was calculated for each trial.

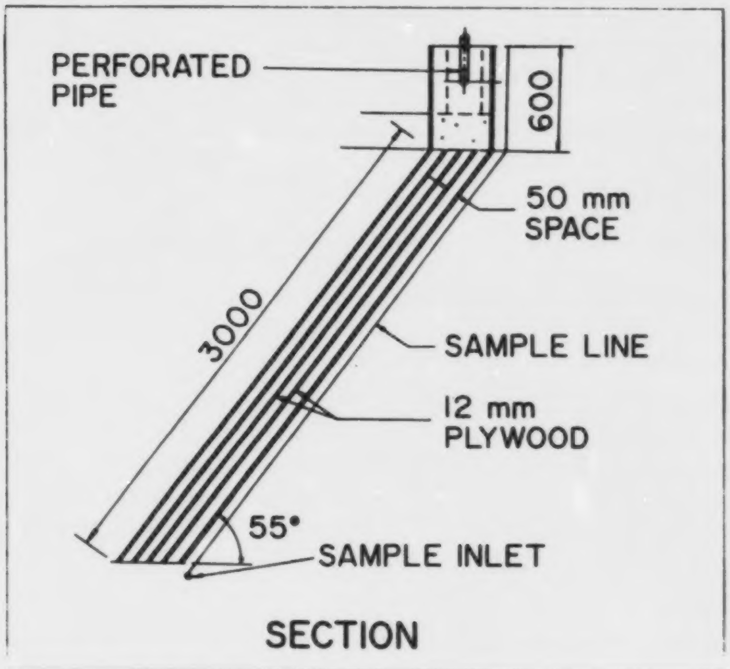
The pilot plate settler apparatus was installed at the beginning of the sedimentation basin. The unit consisted of four 3-metre long, 600-millimetre wide plates, spaced 50 millimeters apart. The plates were constructed of coated marine plywood, 12 millimeters thick and were inclined at an angle of 55 degrees. A 12 millimetre diameter sampling line, two withdraw samples of influent water to the plate settler, was installed below the bottom plate and extended 150 millimeters beyond the inlet to the unit. This was connected to a small centrifugal booster pump. A diagram of the unit is shown in Figure 2 below.

Flow through the pilot unit was obtained by connecting a centrifugal pump to the effluent line. A valve and meter on the pump discharge allowed the adjustment and measurement of various flow rates. The flow rate being tested was allowed to stabilize and then suspended solids measurements were conducted from influent and effluent sampling.

Extensive data was collected at various overflow rates during a one year period. Water temperature and optimum chemical dosages varied. The optimum overflow rate to achieve a suspended solids removal of 70 to 80% was evaluated. If the overflow rate was increased above this value, a critical rate would be obtained where the solids removal performance of the plate settler decreased dramatically. The velocity at this point exceeded the critical settling velocity of this light fragile floc.

FIGURE 2

Pilot Plate Settler



As a result of the pilot testing of the plate settlers, it was recommended that plate settlers be installed in the new plant expansion. An overflow rate slightly less than the optimum obtained in the pilot test was recommended. The installation of plate settlers resulted in an overall plan area reduction and an $800,000 or 20% cost saving in the construction of the settling tanks.

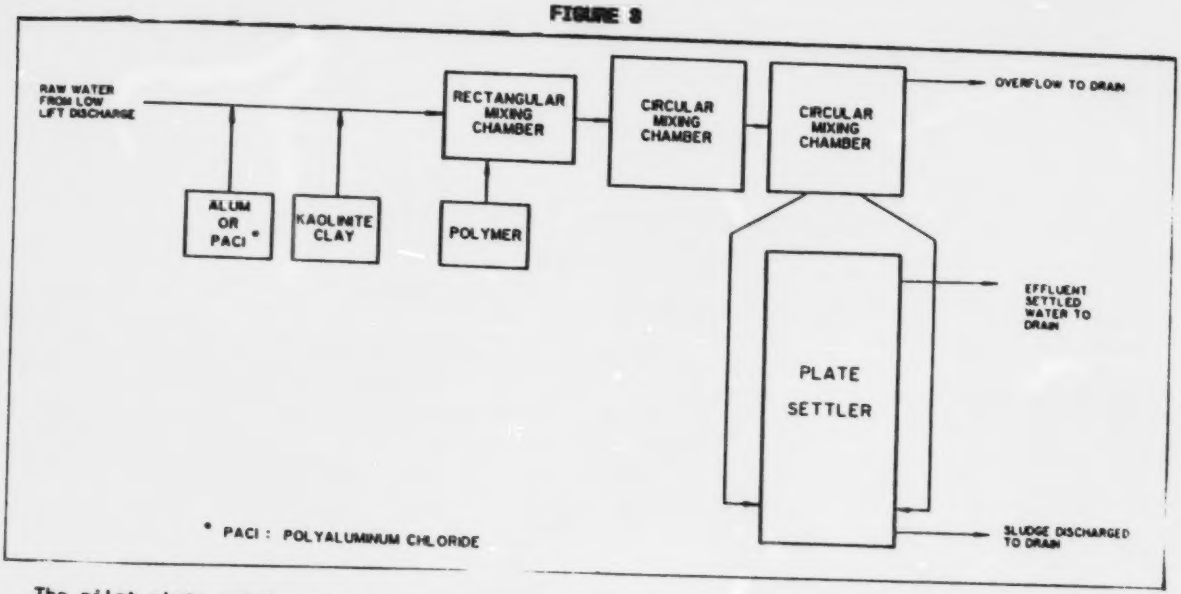
Pilot Testing at a Turbidity Removal Water Treatment Plant

The raw water at this plant is not coloured and the turbidity can range from 1 to over 50 Nephelometric Turbidity Units (NTU). A floc is generated by adding alum to the water and passing the water through flocculation chambers. The existing plant was originally constructed as a direct filtration plant with the intention of adding settling tanks when needed. This philosophy allowed the delay of construction of the settling tanks for 10 years. Interestingly, the original concept in the mid 1970's was to add conventional settling tanks. However, by the mid 1980's when it was time to add settling, plate settlers were coming into use in a number of plants and where pilot tested and eventually selected for this plant.

The overall purpose of the pilot testing was to document the performance of the plate settlers. The testing was done to confirm that the design overflow rate would produce a settled water containing suspended solids of 12 mg/L or less for raw water turbidities of up to 50 NTU. The efficiency of the plate settler was evaluated in the same manner as the other pilot installation; suspended solids measurements of the influent and effluent waters were conducted.

A self contained pilot apparatus was designed and built for this testing program. The apparatus was set up in the low lift pumping station and raw water was fed from the raw water header. A process schematic of the pilot testing is shown below in Figure 3.

FIGURE 3


RAW WATER FROM LOW LIFT DISCHARGE
RECTANGULAR MIXING CHAMBER
CIRCULAR MIXING CHAMBER
CIRCULAR MIXING CHAMBER
OVERFLOW TO DRAIN
ALUM OR PACl *
KAOLINITE CLAY
POLYMER
EFFLUENT SETTLED WATER TO DRAIN
PLATE SETTLER
SLUDGE DISCHARGED TO DRAIN
* PACl : POLYALUMINUM CHLORIDE

The pilot plate settler apparatus consisted of three parallel plates, 3.0-metres long by a 0.6-metres wide, spaced 50 millimeters apart. The plates were encased in a rectangular box. Both the plates and the box were constructed of clear PVC plastic.

The flow through the pilot unit was varied by adjusting a valve position. The flowrate being tested was allowed to stabilize and suspended solids measurements were conducted from influent and effluent sampling.

The pilot plate settler unit itself introduced detrimental conditions which would not be present in a full-scale plant situation. Flocculated water entered the pilot unit under turbulent conditions and some of the floc was sheared into smaller pieces. A polymer was added in the pretreatment stage to strengthen and enlarge the floc and to reduce the shearing of the floc in this turbulent area. In addition, the settled sludge had to pass through this turbulent area on its way to the bottom of the unit, which caused some re-entrainment with the incoming flocculated water.

The floc shearing, sludge settling and settling sludge re-entrainment phenomena were visible through the clear PVC material from which the pilot plant was constructed. It is important tc understand and be able to visualize these phenomena in order to optimize the design of plate settlers.

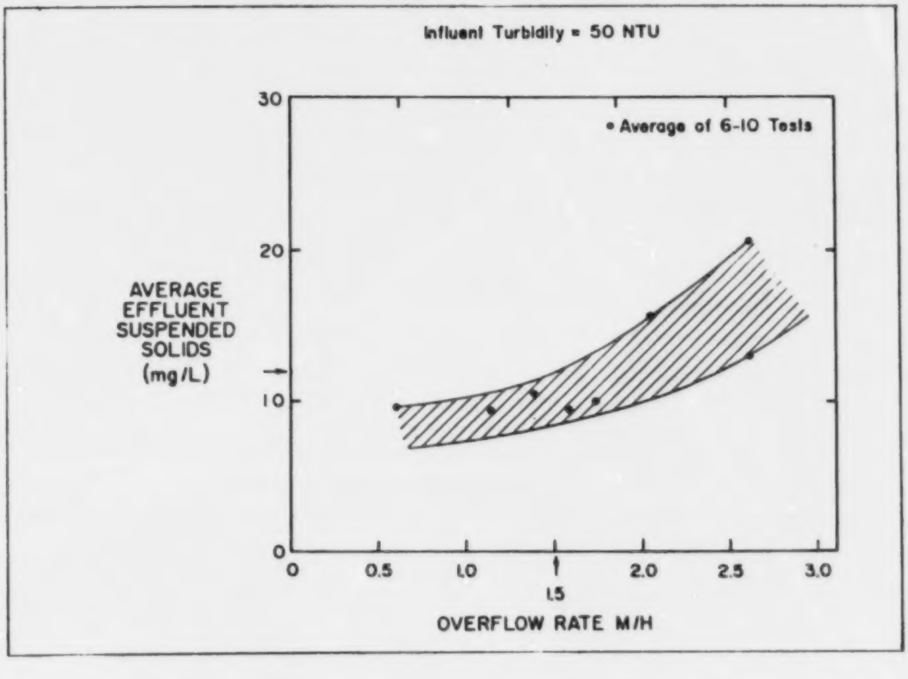
A systematic set of data was compiled for various overflow rates, rate water conditions and chemical dosages. The effluent suspended solids concentration gradually rose as the overflow rate increased. A critical overflow rate where the solids removal performance decreased dramatically was not experienced with this pilot testing. This result is contrary to the results from the pilot testing at the colour removal plant.

In the turbidity removal pilot plant a maximum overflow rate of 1.5 m/h was determined for which the effluent suspended solids was 12 mg/L or less. Figure 4 is a plot of effluent suspended solids versus overflow rate for the turbidity removal pilot plant.

As a result of the pilot testing of the plate settler apparatus, the design overflow rate required to obtain the desired effluent suspended solids was established. The installation of plate settlers in the settling basins significantly reduced the overall plan area of the tanks, and reduced the construction costs by approximately 10%.

FIGURE 4

Effluent Suspended Solids vs. Overflow Rate



Summary

Plate settling tanks can reduce detention time to 20 minutes or less from the 2-4 hours normally used in conventional settling tanks. This short detention time and resultant smaller tank volume can result in lower construction costs. However, there can be problems associated with these shorter detention times. In the United States there is a requirement to have a certain "CT" within the treatment process. When plate settling is used the detention time is reduced which reduces the "CT". The end result could be that additional time might have to be constructed in the form of a contact tank or reservoir thus eliminating the savings discussed above.

The design of a plat settling process is water quality specific, similar to any other settling process. In other words, the design overflow rate will very depending on the nature and quantity of solids to be removed.

The design overflow rate for a plate settling process is lower on an unit area basis than for conventional settling due to interference between upward flowing water and downward moving settled particles within the small 50 millimetre space between plates.

Plate settling should be considered and evaluated as an option wherever settling is required in water treatment.

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MANAGEMENT OF DRINKING WATER PRODUCTION AT THE CITY OF EDMONTON'S WATER TREATMENT PLANTS

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PREAMBLE

The water works industry in North America includes treatment facilities that are efficiency run, sparkling clean and product excellent water, as well as facilities that are run down, poorly managed and obviously not kept in good repair.

The operation and maintenance of a treatment facility should be of concern to everyone with a vested interest, including the consumer, the public officials involved in the administration, and of course, the employees who are responsible for the operation.

Operations Management deals with the "Production" side of all types of industries. Drinking water treatment facilities in most cases are municipal operated water 'factories". In water treatment, Operations Management involves the coordination of various activities that include, but are not limited to: production and inventory planning, personnel administration, engineering, quality control and assurance, data processing, budget and expenditure control. While the general concepts and principles of Operations Management are universal, each organization must adapt these concepts and principles to apply to its own particular situation.

This paper provides a brief overview of Operations Management activities at the City of Edmonton's Water Treatment Plants.

INTRODUCTION

The City of Edmonton is located in Alberta, Western Canada. Edmonton was established as a fur trading post in 1795 and incorporated as a town in 1892. By 1905, it had been granted City status and named Capital of the Province of Alberta.

In terms of area, Edmonton is Canada's second largest City and it boasts the world's largest 'fun, fashion and indoor entertainment centre", known as West Edmonton Mall. Oil, petroleum, natural gas and coal are the City's main industries.

Edmonton's climate is dry with an average rainfall of 345 mm and snowfall of 1350 mm annually. Temperatures vary from $-40^{o}C$ to $+32^{o}C$.

The metropolitan water system serves approximately 700,000 customers within the City and neighboring communities. The Edmonton water system consists of:

1) Two water treatment plants: Rossdale Water Treatment Plant, with a capacity of 250 ML/d (Megalitres per day) and E.L. Smith Water Treatment Plant with a capacity of 180 ML/d.

2) Twelve water storage reservoirs and pumping facilities (total gross storage of 800 ML/d).

3) A 2600 km network of transmission and distribution mains.

WATER QUALITY

Edmonton's source of raw water supply is the North Saskatchewan River which originates from the Saskatchewan glacier, located 500 kms south-west of Edmonton in the Rocky Mountains.

The North Saskatchewan River is subject to variable quality inflows and seasonal conditions which cause fluctuating raw water quality (see Table 1). During the spring run-off the river is subjected to excessive organic loading and poor raw water quality is experienced (colour > 70 T.C.U.; Turbidity > 250 N.T.U.). In the winter time when the river is covered with ice, good raw water quality conditions exist (colour < 5 T.C.U.; Turbidity < 15 N.T.U.). Through most of the winter, river water temperature averages 0.5°C. These extreme river water quality conditions make its treatment a challenge for production staff. Generally, however, treated water quality meets stringent objectives that exceed both the federal and provincial drinking water guidelines (see Table 1). The only exception to meeting aesthetic quality standards occurs occasionally in the spring when organic material carried into the river by snow melt causes taste and odour complaints.

TABLE 1

City of Edmonton Water Treatment Plants
Water Quality Summary

PARAMETERS	RIVER QUALITY		TREATED QUALITY		MAXIMUM STANDARD
	MIN	MAX	MIN	MAX	
Temperature (oC)	0.3	21	1	20	-
Turbidity (N.T.U.)	1.5	750	0.05	0.35	5
Color (T.C.U.)	1	80	1	2	15
Fecal Coliforms (100 ml)	40	160	0	0	0
Hardness (mg/l)	130	202	100	125	-
Alkalinity (mg/l)	102	172	37	78	-
Fluoride (mg/l)	0.1	0.2	0.9	1.1	1.2
THM's (ug/l)	0	0	0	6	350

WATER TREATMENT

To meet the City established treated water quality objectives, various water treatment processes, unit operations and chemicals are required.

Tables 2 and 3 summarize the water treatment processes, unit operations and chemicals used in the production of drinking water at the two Water Treatment Plants.

TABLE 2

City of Edmonton Water Treatment Plants
Treatment Processes and Chemicals

PROCESS	*TREATMENT*	*CHEMICALS*
ADSORPTION	- Removal of dissolved organic matter (i.e. color, and taste and odor causing compounds)	Powdered Activated Carbon
CLARIFICATION	- Removal of suspended colloidal, organic microbiological and other matter	Alum / Polyelectrolyte
DISINFECTION	- Kills bacteria, viruses and other micro-organisms	Chlorine dioxide / Chlorine / Ammonia
FLUORIDATION	- Dental care for children	Hydrofluosilicic acid
SOFTENING	- Reduction of hardness	Lime / Soda Ash
RECARBONATION	- pH control and water stabilization	Carbon Dioxide
FILTRATION	- Particulate and microbiological removal through sand/anthracite media	-
WASTE HANDLING	- Waste lime sludge volume reduction and disposal	-

TABLE 3

City of Edmonton Water Treatment Plants

Unit Operations

Unit	*Function*
Intake Structure	- Receives water from the river
Low Lift Pumps	- Transfers water from intake to Plant
Clarifier 1	- Remove suspended, microbiological, and organic matter from water
Clarifier 2	- Remove hardness from water
Filters	- Remove colloidal and microbiological matter from water
Reservoirs	- Stores water up to 20 hours
High Lift Pumps	- Distribute water to city
Thickener	- Thickens lime sludge from clarifier waste
Centrifuge	- Further extraction of water from lime sludge

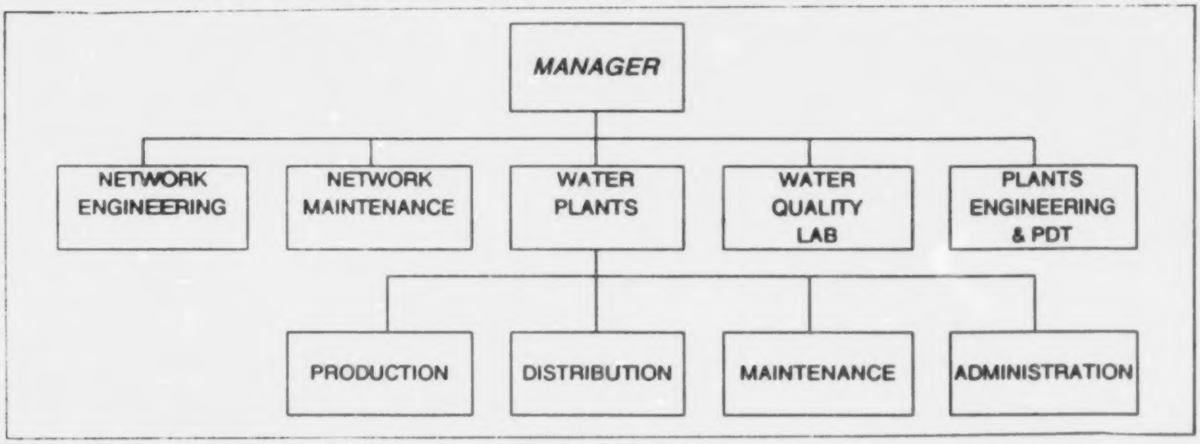
PRODUCTION MANAGEMENT

The City of Edmonton Environmental Services Department is divided into 3 main branches: (1) Water; (2) Drainage; and (3) Solids Waste. Figure 1 summarizes the organization chart for the Water Branch.

FIGURE 1

The City of Edmonton

Environmental Services Department Water Branch


MANAGER
NETWORK ENGINEERING
NETWORK MAINTENANCE
WATER PLANTS
WATER QUALITY LAB
PLANTS ENGINEERING & PDT
PRODUCTION
DISTRIBUTION
MAINTENANCE
ADMINISTRATION

The operation of the two water treatment plants is the responsibility of the Water Plants Section of the Water Branch. Figure 2 depicts the general philosophy adopted in the Management of drinking water production at the two water plants. The production operations group is ultimately responsible for the production of drinking water. To enable the production operation group to fulfil this mandate successfully, various service groups have been established to provide support services to the production operations group. Figure 3 shows the organizational structure of the production operation group and summarizes staffing for the support groups.

FIGURE 2

Pictorial Representation of
Management Philosophy at
City of Edmonton's Water Plants

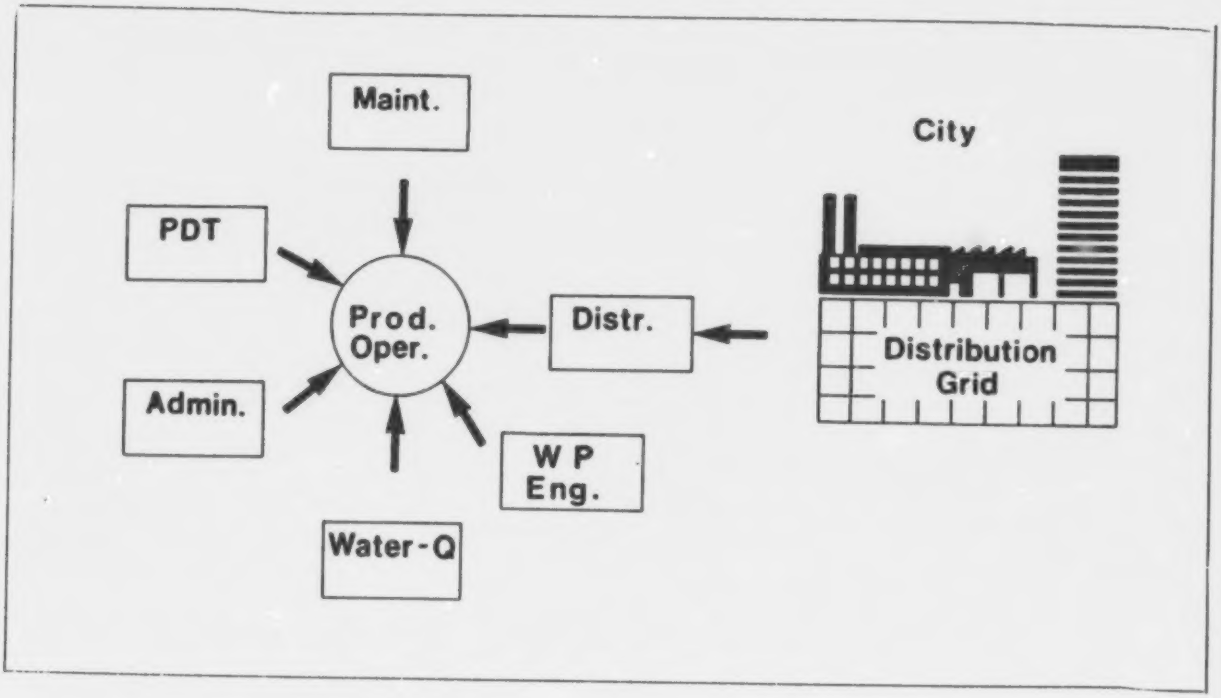
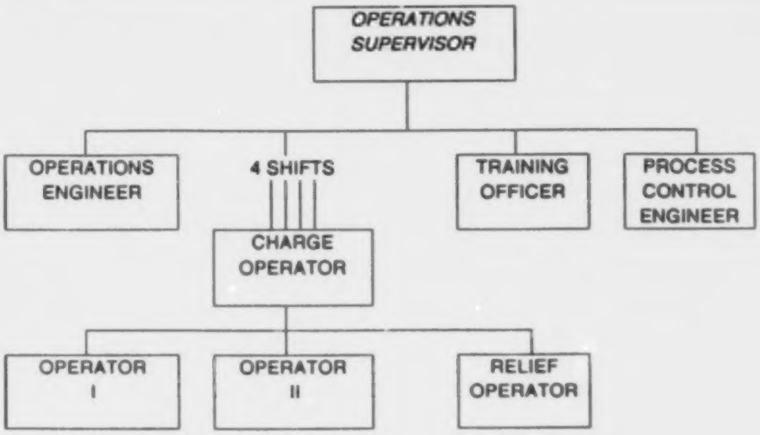


FIGURE 3

Environmental Services Department
Production Operations Group
(At Each Plant)


OPERATIONS SUPERVISOR
OPERATIONS ENGINEER
4 SHIFTS
TRAINING OFFICER
PROCESS CONTROL ENGINEER
CHARGE OPERATOR
OPERATOR I
OPERATOR II
RELIEF OPERATOR

OTHER SUPPORT GROUPS STAFFING

MAINTENANCE GROUP
(at each Plant)

- Supervisor
- Mechanical Lead Hand
- Electrical Lead Hand
- Millwrights (2)
- Instrument Techs. (2)
- Electricians (2)
- Plumbers (1/2)
- Welders (1/2)
- Utility Helpers (2)

PLANTS ENGINEERING GROUP
(one group for both Plants)

- Chief Engineer
- Senior Project Engineers (2)
- Intermediate Project Engineers (2)
- Junior Project Engineers (2)
- Engineering Technicians/ Inspectors (5)
- Clerks (2)

WATER QUALITY (LAB)
(one group for both Plants)

- Director
- Organic Scientist
- Inorganic Scientist
- Microbiology Scientist
- QA/QC Officer
- Laboratory Supervisor
- Laboratory Technicians (5)
- Laboratory Assistants (2)

DISTRIBUTION
(one group for both Plants)

- Transmission Engineer
- Distribution Technician
- Distribution Operators (5)

PROCESS DEVELOPMENT TEAM
(one group for both Plants)

- Director
- Research Engineer
- Research Scientist
- Research Technician

ADMINISTRATION
(one group for both Plants)

- Office Administrator
- Purchase Officer
- Storesman (2)

PRODUCTION OPERATIONS ACTIVITIES

At each of Edmonton's two Treatment Plants, the Production Operations Group has the overall responsibility for the production of drinking water. Detailed activities of the production operations groups are:

[I] **Water Production:** As part of the water production activities, the group is responsible for the control of the treatment process and operation of all equipment related to these processes. The group has to perform the necessary water quality analyses, make appropriate equipment operating decisions and meet quality and quantity objectives. The quality and quantity objectives are summarized under Table 4.

TABLE 4
City of Edmonton Water Treatment Plants
Productions Group
Quality & Quantity Objectives for Treated Water

PARAMETERS	MAC *	OBJECTIVE	OPERATING RANGE
Turbidity (N.T.U.)	1	< 0.35	< 0.25
Color (T.C.U.)	15	< 5	< 2
pH	8.5	8.0 - 8.5	8.2 - 8.4
Agressive Index	N/A **	> 11.5	> 11.5
Taste and Odor	N/A	Inoffensive	Inoffensive
Total Chlorine (mg/l)	> 1.8	1.8 - 2.4	1.9 - 2.2
Total Oxidants (mg/l)	0.5	0.2 - 0.5	0.3 - 0.4
Fluoride (mg/l)	0.8 - 1.2	0.9 - 1.1	0.9 - 1.1
Total Hardness (mg/l)	N/A	105 - 125	110 - 120
T Coliforms (100 mls)	10	0	0
F Coliforms (100 mls)	0	0	0
Trihalomethanes (mg/l)	350	< 10	0
Chlorine dioxide yield (%)	0.85	> 85%	95 - 100%
On-Site storage (%)	-	> 65%	> 75%

* Maximum Acceptable Concentration
** N/A - Not Applicable

[II] Overall Coordination: As part of its activities, the production operations group is responsible for overall plant coordination. This includes coordinating with the support groups (next section), and external agencies (eg. contractors, consultants, provincial regulators, etc.) to ensure the safe and reliable production of drinking water.

[III] Applications Engineering: The production operations group activities also includes treatment process engineering such as process problem solving, optimization, operational strategies, computer control applications, minor design changes, information gathering, record keeping and report generation.

[IV] Operator Training: Training is an essential part of any production team. Training of operators impacts on the level of service provided to the public. At the City's water treatment plants, the operator training program consists of three levels of training. These 3 levels have been designed to correspond to the 3 levels of provincial operator certification and the 3 levels of operator responsibilities at the plant.

Level I: This level is divided into two parts. The first part deals with on-the-job training to perform water quality analyses, operation of various equipment, and safety aspects. The second part deals with the basic knowledge of support systems, eg: hydraulics, electricity, instrumentation, etc.

Level II: The second level of training is aimed at training operators in making process decisions, such as increasing or decreasing chemical feeds, troubleshooting process problems, etc. The training introduces the operators to physical and chemical aspects of water treatment.

Level III: The third level of training exposes the operator to the supervising and managing aspects of operations. Supervising of subordinates, public relations, company philosophy, financial management are some of the topics covered under this level.

PLANT SUPPORT GROUPS

To assist in the successful operation of a water treatment plant, various service support groups are essential. Some form of formal or informal communications systems has to exist between the service groups and the production group to bring order to the "jungle" of operations activities that need to be carried out on an hour to hour basis. The following are the support groups and their functions at the Water Treatment Plant.

[I] Maintenance Group: Good maintenance ensures effective production operations. The maintenance group ensures, through a formal maintenance program, that minimum equipment downtime occurs. Because machines and buildings continually depreciate, they need repair and, on occasion, replacement. A good maintenance program enables the production operations group to consistently meet their quality and quantity requirements.

[II] Water Quality/Laboratory Group: In a drinking water production plant, public health safety is the absolute priority. Therefore a water quality group is essential. The water quality group at the Water Treatment Plant trains and provides production operators with the necessary skills to analyze water in order to meet drinking water quality objectives. In addition, the group audits (inspects) the performance of the operators on a random basis, as part of its quality assurance program. When new analyses, or more stringent quality requirements are introduced, the water quality group develops and assists in the implementation of new water quality analyses.

(III) Distribution Group: Produced quantities in any industry are based on the demand and sales of the product. In drinking water treatment the situation is similar except the demand is influenced by the weather and the time of day. Minimum "sales" are guaranteed due to the monopoly of the market. Day-to-day, the distribution group forecasts demand based on a historical model, on a day to day basis and determines production quota's for the plants. The Distribution Group also coordinates distribution or production interruptions due to system failures, quality shortcomings, planned maintenance, etc.

(IV) Process Development Team (P.D.T.): Drinking water technology, like technology in most industries, is advancing rapidly. Developments in instrument technology have enabled detection of contaminants (chemical or microbiological) at very low concentrations. The scientific knowledge of potential health effects of these contaminants is also increasing. Therefore new ways of water quality monitoring and treatment have to be developed on an ongoing basis. Research and development work is necessary to determine new and better ways of ensuring safe drinking water quality. The P.D.T. group is dedicated to provide the research and development support to the Production Operations Group. The PDT is currently studying the taste and odour problems experienced during the springtime runoff conditions.

(V) Plants Engineering Group: The production operations group and all of the above technical groups require engineering support at all times. Water Plants Engineering is responsible to translate concepts into reality through the development, planning, implementation and management of capital projects related to Water Treatment Plants, Water Storage reservoirs and pumping facilities. This activity is accomplished by either in-house engineering teams or through outside consulting engineering firms. All capital works are developed in line with the Water Branch philosophy to provide drinking water: (i) that exceeds all quality and aesthetic standards, (ii) that is supplied reliably in adequate quantities, (iii) that gives customer value for money and (iv) whose production and distribution activities minimize the impact on the environment. The engineering group also provides ad-hoc specialized emergency engineering services to the maintenance, distribution and production groups, (i.e. major repairs to mechanical, structural, electrical, etc., equipment). The group also coordinates recruitment of specialized consultants and provide contract administration/project management services for all of the other groups.

(VI) Administrative Group: This is the only non-technical support group, at the Water Plants. However, it is just as essential and a vital service to the productions group. Budgeting and financial control is an important part of the overall production of drinking water. The Administrative Group assists in the preparation and monitoring of the annual operating budget for all groups at the Treatment Plants. The group is also responsible for preparing monthly and annual reports for regulatory agencies, public information and upper management reviews. Finally the group provides contract and material procurement and material inventory planning services to all groups.

The six support groups mentioned are an integral part of drinking water production at the Plants. These groups are essential in the production of drinking water.

LABOUR RELATIONS

A healthy labour/management relationship is instrumental in maintaining and improving productivity. Open communication and good rapport between the staff, unions and management can help in areas such as: staff qualifications and training, flex hours of work, incentive pay systems, work activities design, etc. At the Plants, regular discussions are held with the staff and their unions which have, over the course of the last few years, resulted in better trained and qualified operators, decreased unplanned absences and enabled an increase in operator responsibilities. This rapport has also helped improve employee morale in the Plant and consequently performance and water quality.

SAFETY

At any work site, management has the responsibility of also providing safe working conditions for its employees. However, people are still injured on the job, so the effort to reduce accidents becomes the unending responsibility of management. The safety record of the municipal water or wastewater industry in North America is poor. Only the mining industry has a worse record. The cost of accidents in terms of both human suffering and money is high, so worker safety is a high priority.

A formal safety program is in place at the two water plants. The program addresses safety issues through a committee approach. Four safety committees exist, each committee has a specific mandate and responsibilities:

(i) Safety Concern and Hazard Recognition Committee: This committee includes all plant staff (operators, maintenance workers, support staff and middle management) who meet once a month to discuss safety concerns or identify safety hazards on the job. Action is taken immediately through either work order request, procedure change, budget requests, etc.

(ii) Safety Promotion and Awareness Committee: This committee, as the name suggests, promotes safety consciousness among the employees through monthly safety films, posters and news letters.

(iii) Safety Training Committee: This committee is responsible for providing all aspects of training to employees. Training programs such as New Employee Safety, Scott Air Pack Use, Fork Lift Use, Emergency Response Procedures, Dangerous Goods Acts, etc. are offered by the committee on an ongoing basis.

(iv) Central Committee: This committee meets quarterly and consists of middle and upper management and chairpersons of each of the other committees. The central committee reviews the activities of the other committees, provides general direction to the committees and makes senior level decisions on safety policies and procedures.

The above approach to safety has enabled maintenance of good safety records at the plants.

PUBLIC RELATIONS

Municipal facilities belong to the public-at-large who, through taxes or water rates have invested considerable sums of money to construct, operate and maintain facilities. Municipal employees are public servants and are responsible to the public for their actions. As part of this philosophy, the City of Edmonton, Environmental Services Department has in place a "3 C's" campaign for its customers (public-at-large). The 3 C's stand for: Courtesy, Concern and Competence. Feedback from the customers is requested through the means of 3 C's cards that are distributed to the customers for their evaluation of the service provided by the employees of the Environmental Services Department. In addition, the department also has a dedicated public relations team that provides public tours, sets up public displays, conducts educational programs and communicates through the news media.

Internally, the department also promotes positive attitude, good morale and employee recognition through formal recognition programs, employee of the year awards, internal newsletter and poster campaigns.

PERFORMANCE STANDARDS

Finally, with all of the above activities in place, some form of performance monitoring is required. Performance standards are highly desirable for almost every kind of organized work. In drinking water production, standards are required for the quality, quantity, cost, energy usage, etc. Once standards have been set, they provide a basis for comparisons in production performance. The standards encourage productivity expectations to be met. Shortcomings will prompt investigations and remedial actions. At the Water Plants, measures of performance have been set for the main technical groups. A summary of annual performance measures at the Water Treatment Plants are shown in Tables 5 through 8 below.

TABLE 5

City of Edmonton Water Treatment Plants
Productions Operations Group - Annual Performance Measures

DESCRIPTION	*OBJECTIVE * 1987*	*ACTUAL * 1987*
QUALITY		
Quality Variances (No)	10	3
Hardness Variances (No)	15	9
Acceptable QA Samples (%)	> 95%	>93%
QUANTITY		
On Site storage < 65% (days)	< 30	15
On Site storage < 75% (days)	< 120	50
PROCESS		
Filter Performance (Avg. Turbidity)	< 0.25 NTU	0.12 NTU
Filter Efficiency (ML/BW)	45	43
Process Losses (% Production)	4%	2%
ENERGY		
Power Consumption (kwh/ML)	191	189
Gas Consumption (GJ)	37,000	38,500
PRODUCTION		
Treated Water (ML)	54,082	55,000
Sludge Waste (Tonnes)	14,000	12,514
Cost ($ / ML)	55	53
PUBLIC RELATIONS		
Tour Visitors (No)	> 1,500	2,070

* For one of the two Water Treatment Plants

TABLE 6

Maintenance Group – Annual Performance Measures

DESCRIPTION	OBJECTIVE * 1987	ACTUAL * 1987
MANPOWER DISTRIBUTION		
Breakdown Maintenance (%)	10	8.5
Scheduled Maintenance (%)	40	46
Preventive Maintenance (%)	10	3.5
Improvement Maintenance (%)	20	6.5
Other (%)	20	35.5
Overtime (%)	2	1.5
MAINTENANCE PRODUCTION		
Total Work Orders (No)	1,200	982
Late Work Orders (%)	15	12.1
Cost ($ / ML)	15.1	14.11

* For one of the two Maintenance Groups

TABLE 7

Water Quality Group – Annual Performance Measures

DESCRIPTION	OBJECTIVE 1987	ACTUAL 1987
Tests Performed (No)	69,000	70,000
Samples Tested (No)	20,000	22,000
Tests / Hour	> 3	3.5
Cost ($ / Test)	9.8	10
Acceptable QA Samples (%)	95	98

TABLE 8

Distribution Group – Annual Performance Measures

DESCRIPTION	UNITS	OBJECTIVE 1987	ACTUAL 1987
System Pressure < 240 kPa	instances	< 500	595
System Pressure > 700 kPa	instances	< 5	0
Storage Volume < 65 %	days	< 20	15
Energy	kwh/ML	<420	440
Chlorine Residual < 1 mg/l	instances	0	2
E.L. Smith Plant Production	% Total	47	48
Rossdale Production	% Total	53	52

FINANCIAL MANAGEMENT

Management and operators share the responsibility of controlling and managing the costs of municipal facilities. Annual budgets for labour, materials, repairs, equipment, contractors, and other expenditures determines the total amount of money that should be allocated for the system operation. In addition, any costs for debenture borrowing to construct the treatment and distribution facilities should also be included in the overall operating cost. Figure 4 provides a breakdown of annual costs for operating the Water Treatment Plants. It must be noted that the costs of drinking water production is significantly influenced by the treatment requirements. A comparison of treatment process versus water rates in various cities in Canada are summarized in Table 9. Additional treatment processes result in increased capital expenditure which in turn increases operating costs, i.e. chemicals, energy, staff, etc. Figures 5 to 6 summarize, in pie chart form, the Water Branch operating expenses and revenue requirements.

FIGURE 4

City of Edmonton
Water Treatment Plants
Operating Expense

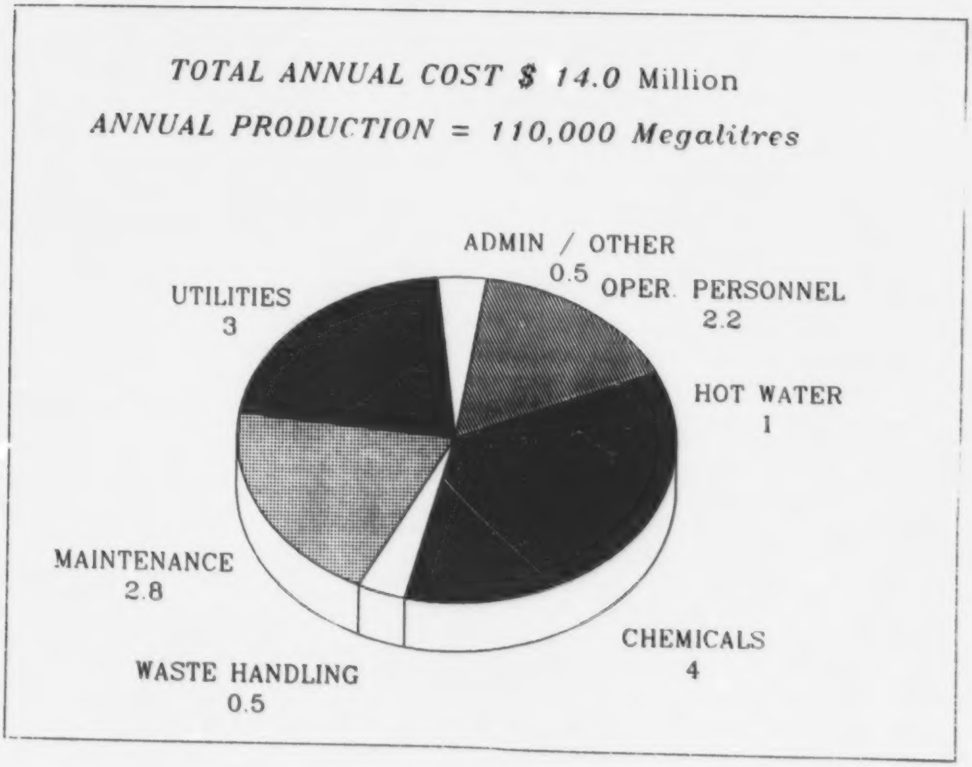


TABLE 9

Canadian City Water Rates Comparison

Description	*Edmonton*	*Vancouver*	*Calgary*	*Winnipeg*	*Toronto*
*Monthly water rates **	$21.15	$10.42	$18.30	$11.74	$13.11
Water source	River	Reservoir	Res./River	Lake	Lake
Treatment Process					
Adsorption	Yes	No	No	No	No
Disinfection	Yes	Yes	Yes	Yes	Yes
Clarification	Yes	No	Yes	No	No
Softening	Yes	No	No	No	No
Recarbonation	Yes	No	No	No	No
Filtration	Yes	No	Yes	No	Yes
Waste Handling	Yes	No	No	No	No
Fluoridation	Yes	No	No	Yes	Yes

** Based on 22.7m3 / month consumption (Oct '88)*

FIGURE 5

City of Edmonton
Water Branch Operating Expenses

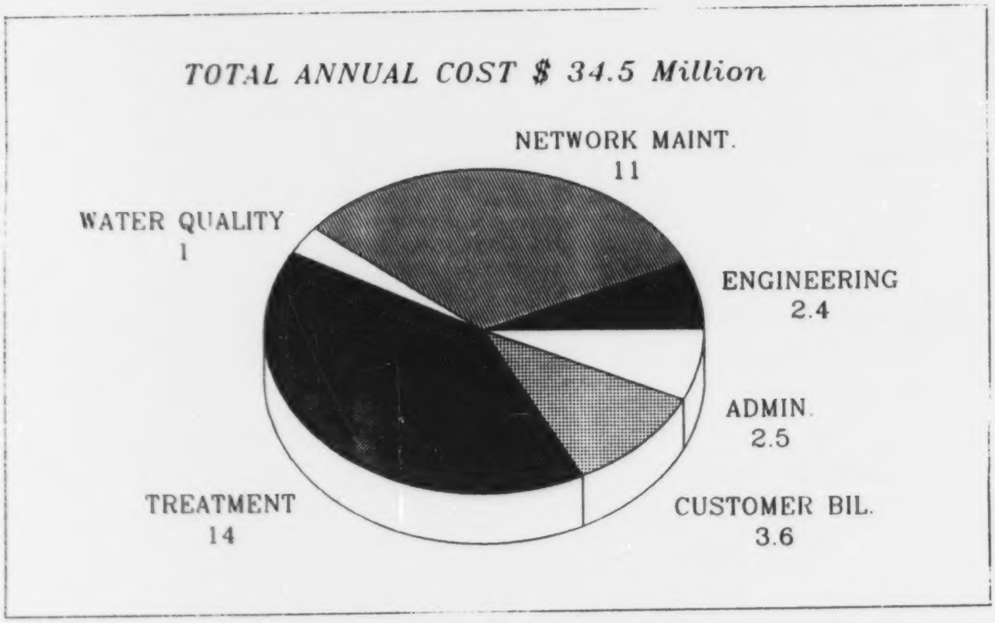
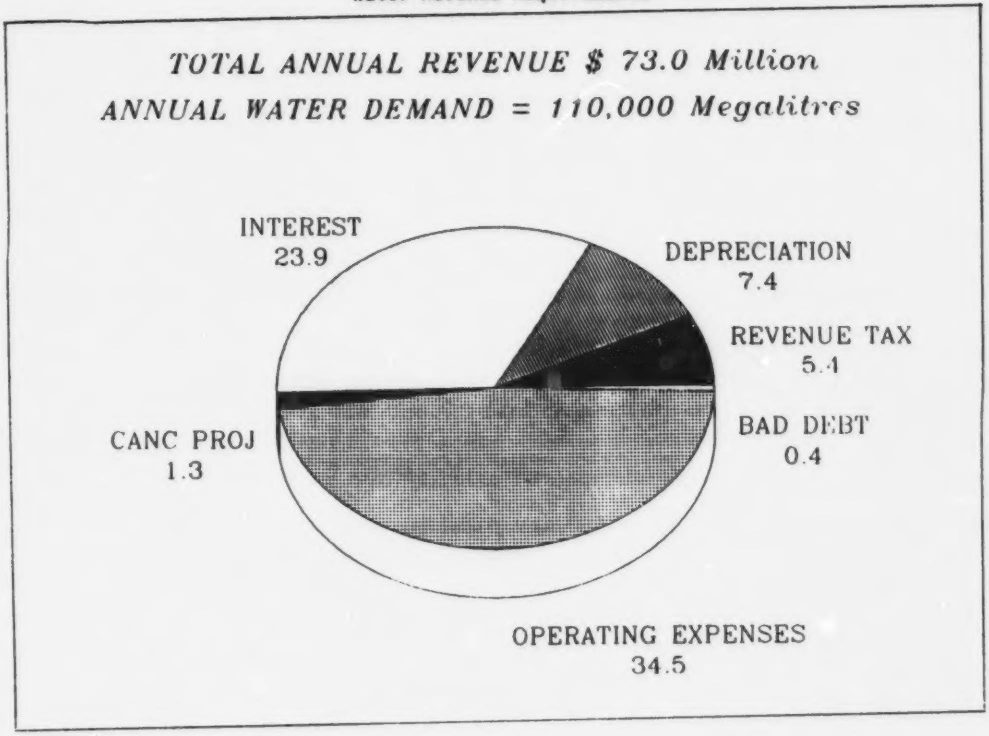


FIGURE 8
City of Edmonton
Water Revenue Requirements



CONCLUSION

This paper and presentation is intended to provide an overview on the Production Management of drinking water at the Water Treatment Plants in Edmonton. Note that the costs and required levels of activities are dependent upon the magnitude of operation, i.e. capacity, treatment processes, etc. The Edmonton costs are indicative of the variable water quality of the North Saskatchewan River, the stringent treated water quality objectives adopted by the City of Edmonton are a reflection of the high expectations by the local consumers.

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WATER NETWORK MAINTENANCE TRAINING PROGRAM
The City of Edmonton Environmental Services

Martin A. Hovestad
and
Sanja M. Hugo
Environmental Services Department
City of Edmonton

BACKGROUND

Water and Sanitation, now Environmental Services, under went major reorganization late in 1985.

Water Network Maintenance was created as one section of the Water Branch. The services provided by this section consist of the 24 hour, 7 days a week, dispatch and trouble response to customers and water crews, maintenance and repair of the water distribution system; installation, repair and testing of the water meters, Alberta 1st Call staking services, customer relation and Engineering Support Services.

The entire Branch orientation towards customer satisfaction, and to provide the best service at the lowest possible cost, was strongly emphasized.

After the regrouping and organizing of the Water Network Maintenance functions, it became apparent that a good training program is a must. It was obvious that each group had a different approach to the same type of work. To achieve more for less, we had to find the best way of doing things and utilize all the expertise and knowledge from our best employees to unify the work process, and attain performance consistency.

The development of the productivity monitoring system and defining task times took first place. This was followed by development of the procedures manual and training program that matches the productivity monitoring system.

The first system was developed in house, and it took a lot of time and cooperative effort of everyone to have it completed, and to be user friendly.

At an early stage we realized that we did not have the manpower resources to prepare a good procedure manual and training program, and that we would have to utilize some external forces by using a contractor.

At that stage we prepared the estimate and time schedule required to complete this work. Proposals from various contractors were received, and after detailed interviews and analysis we selected the SP3 Systems Co. to work on this project. To have this work successfully completed we committed to provide ample time from our employees, be it field or supervisory staff. Also, we developed and filled the position of the Training Foreman that was selected in cooperation with the contractor. The Training Foreman was available on a full time basis to the contractor during the final stages of the project, and when through the training process to become a trainer.

SP3 were required to research and select the best, safest and most productive methods for each job function by evaluating our existing work process, investigating what other cities are doing, checking on the existing AWWA standards and recommendations, and also involve an Engineering expert to evaluate the proposed method for each work task. Only by compiling all this information and selecting the best and safest method we felt that our cost effectiveness and customer satisfaction will be met.

This entire process was new to our outside Union workers and was, in the beginning, treated with suspicion. All kinds of rumors were floating around, such as this is being done in order to pick the employee's brains and then use the document for private contractors, which would cause loss of jobs, and similar negative feelings were evident. However, with positive enforcement and eventually after the hiring of the Training Officer from the Foremans' group, the attitude towards the project changed and everyone started to contribute fully to the process.

The training program was adopted as their program and the pride over accomplishments took over the initial fear.

The following is the description of the training program.

INTRODUCTION

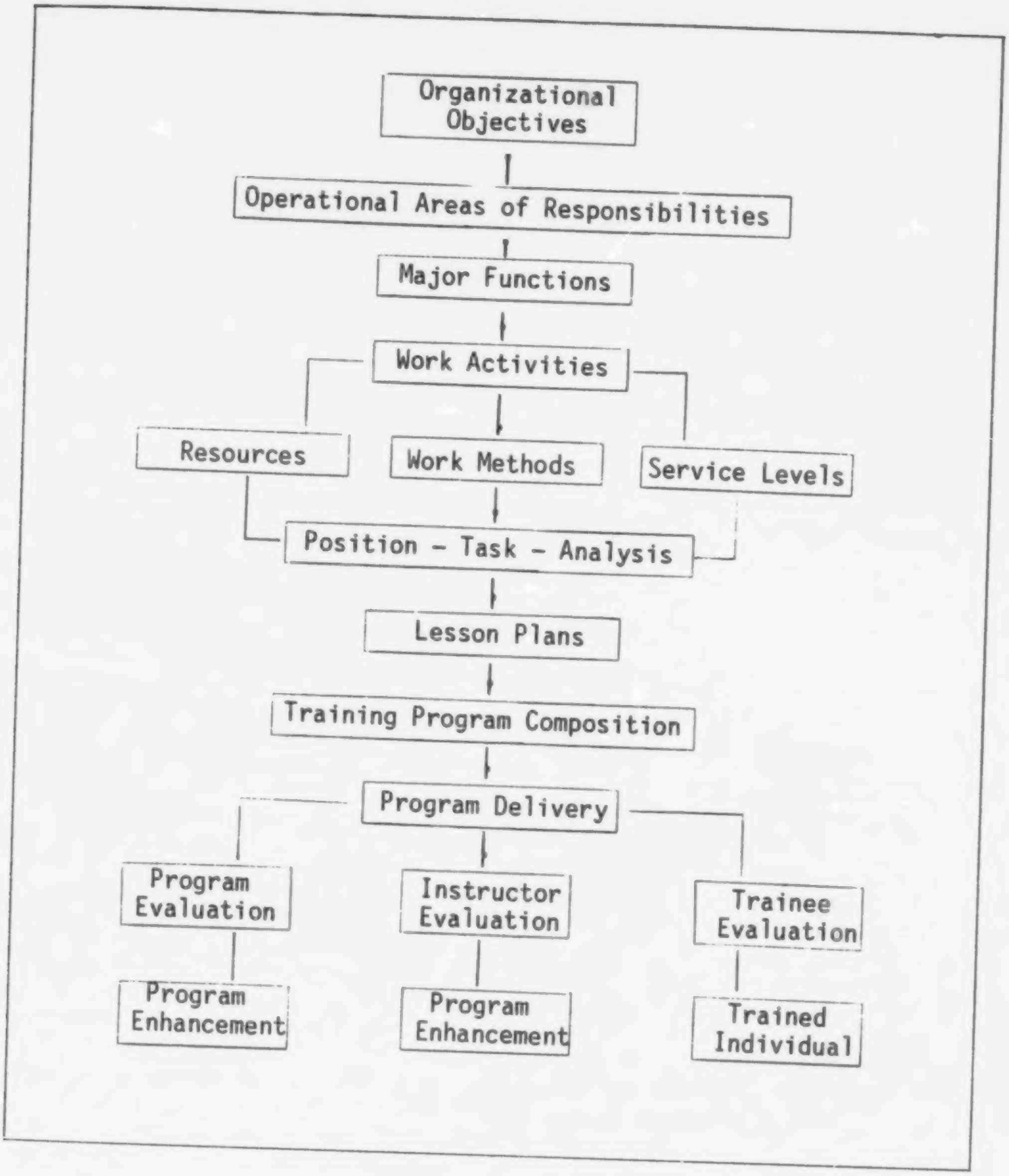
The Edmonton Water Network Maintenance Training Program is a performance-based program that links organizational change, performance development and employee development through a comprehensive training process. The program is designed to help Water Network build a strong organizational climate that reflects the cooperative spirit among employees, service oriented work behaviour, professionalism and high productivity. Management's decision to undertake the custom development of this program has cleared the way for employees to be trained in the best and the safest work methods used in Water Network Maintenance. Performance consistency among all employees is to be raised to the level of those most competent. Flexibility, diversity, skill and knowledge is limited only by individual employee interest. The program provides a powerful opportunity to any employee aspiring to master the full range of functional responsibilities Water Network Maintenance has to offer. Career development in any of these responsibilities is one of the key strategies management wishes to pursue. Water Network offers career challenges in operations control, supervision, technology, communications, services and administration, and this training program helps employees achieve success in their area of choice.

The success of its employees in turn helps Water Network Maintenance in achieving another one of its main goals: good customer relations. The training program is considered to be a major impetus to that goal. Successful customer relations is also viewed, however, as one of the motivations employees will use to establish their pride in high quality workmanship and service.

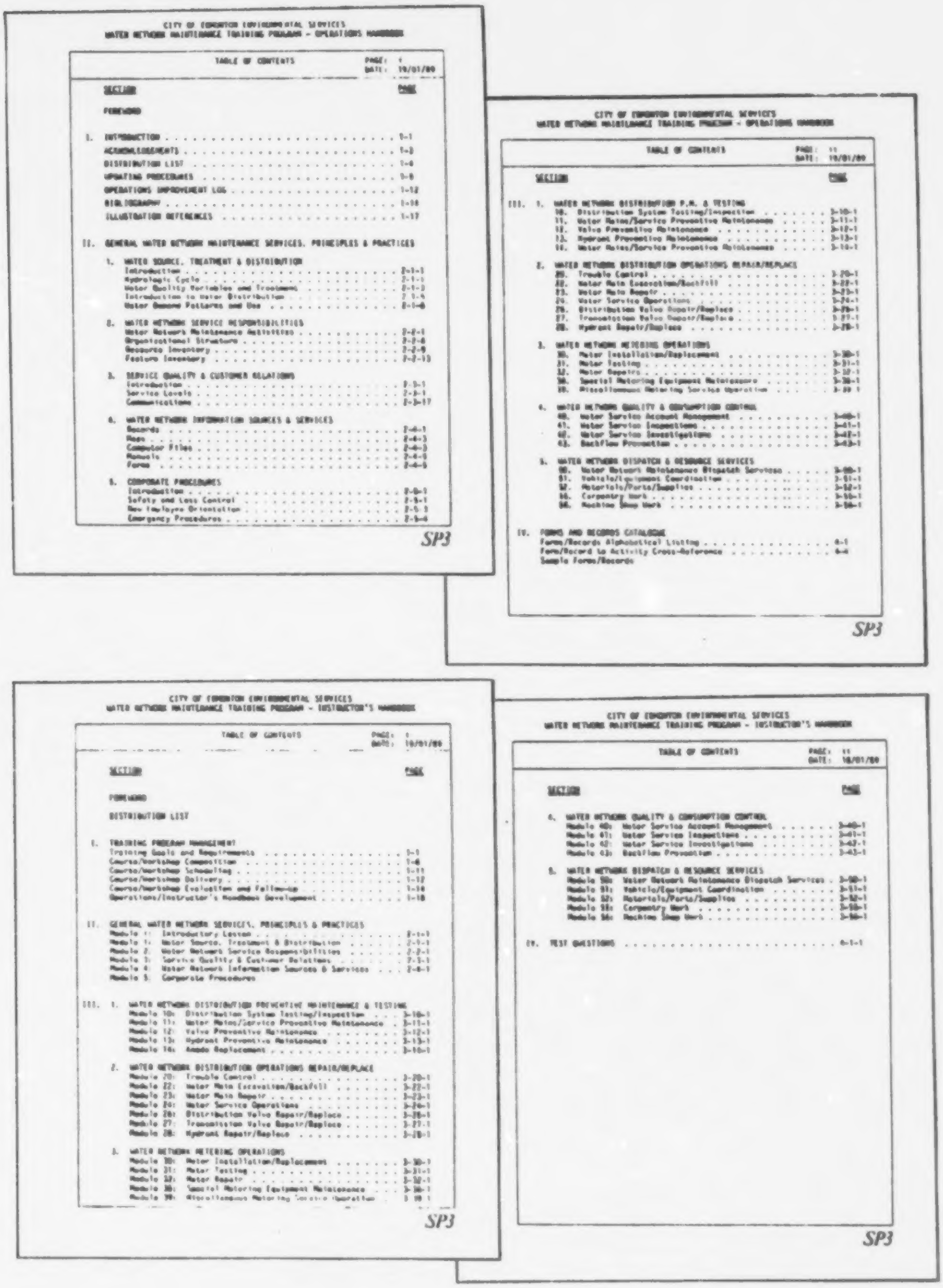
The training management strategy built into this program is designed to continually seek the involvement of employees in incorporating new operations developments and work improvements. Employee involvement, technological development and management style embrace the training process to assist the organization, staff and new employees in their efforts to improve.

PROGRAM DOCUMENTATION

The documentation for the program consists of two handbooks, an Operations Handbook and an Instructor's Handbook. The program is based on actual work activities, and structured in accordance with the following model:


Organizational Objectives
Operational Areas of Responsibilities
Major Functions
Work Activities
Resources
Work Methods
Service Levels
Position - Task - Analysis
Lesson Plans
Training Program Composition
Program Delivery
Program Evaluation
Instructor Evaluation
Trainee Evaluation
Program Enhancement
Program Enhancement
Trained Individual

As shown in the Table of Contents below, the program addresses Water Network's operations activities, its general principles, practices and procedures, and is integrated with other training and safety programs which exist within the Section or the Department.



CITY OF EDMONTON ENVIRONMENTAL SERVICES
WATER NETWORK MAINTENANCE TRAINING PROGRAM - OPERATIONS HANDBOOK

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CITY OF EDMONTON ENVIRONMENTAL SERVICES
WATER NETWORK MAINTENANCE TRAINING PROGRAM - INSTRUCTOR'S HANDBOOK

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TRAINING GOALS AND REQUIREMENTS

The training program is designed around the relationship between Water Network Maintenance goals, and staff training needs. The Section's goals and their priority will have a key impact on the composition and scheduling of courses and workshops. Top priority goals include:

Organizational Oriented Goals:

- Service knowledge development
- Customer relations development

Work Oriented Goals:

- Service oriented work behaviour
- Work quality improvement
- Performance consistency improvement
- Productivity improvement

Employee Oriented Goals:

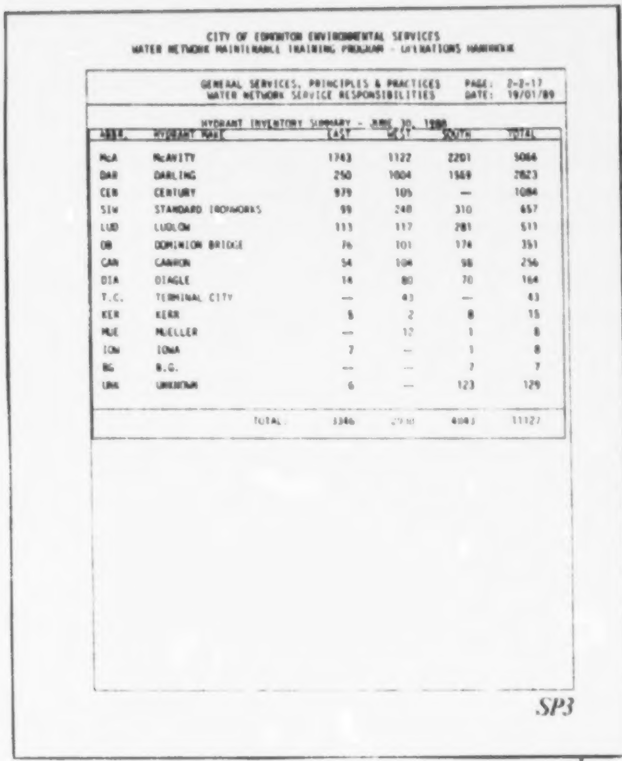
- Job interest training
- Job upgrading training
- Job transfer training
- New employee training

Training can help achieve these goals for the organization as a whole as well as for the career of individual employees. The goals listed above, and those added in the future, are used as the starting point for the composition of flexible training materials to keep courses and workshops dynamic and responsive to operations performance progress.

ORGANIZATION ORIENTED GOALS

Service Knowledge Development

Service knowledge development means providing employees with an understanding of service level quality and quantity descriptions. It is an essential preliminary objective to improving customer relations. Employees who are not aware of the levels of service Water Network Maintenance provides cannot inform a customer, friend or neighbor of the Section's achievements, aspirations and challenges. For example, how many people realize that water main leaks can be pinpointed within one metre of the leak 95% of the time. Water Network employees are unique within Environmental Services as they operate at the front lines, the customer interface, of a complex technological process to provide water, one of life's most essential resources. Their technical skill and knowledge must be augmented with a solid understanding of the capabilities, resources, limitations, service quality and service quantity their Section can deliver. It is therefore that the Section's service level information (see illustration) is included in each and every course and workshop delivered through this training program.



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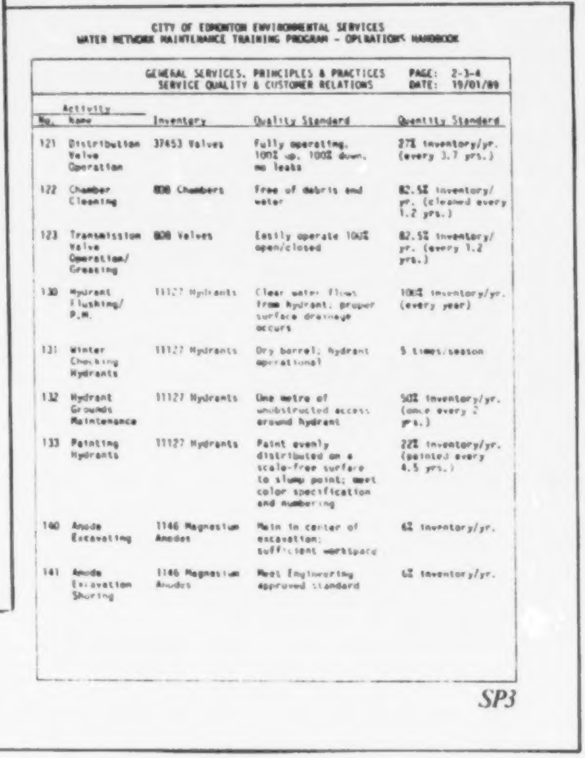
WATER NETWORK MAINTENANCE TRAINING PROGRAM - OPERATIONS HANDBOOK

GENERAL SERVICES, PRINCIPLES & PRACTICES PAGE: 2-2-17
WATER NETWORK SERVICE RESPONSIBILITIES DATE: 19/01/89

HYDRANT INVENTORY SUMMARY - JUNE 30, 1988

ABBR.	HYDRANT MAKE	EAST	WEST	SOUTH	TOTAL
McA	McAVITY	1743	1122	2201	5066
DAR	DARLING	250	1004	1569	2823
CEN	CENTURY	979	105	—	1084
SIW	STANDARD IRONWORKS	99	240	310	657
LUD	LUDLOW	113	117	281	511
DB	DOMINION BRIDGE	76	101	174	351
CAN	CANRON	54	104	98	256
DIA	DIAGLE	14	80	70	164
T.C.	TERMINAL CITY	—	43	—	43
KER	KERR	5	2	8	15
MUE	MUELLER	—	12	1	8
IOW	IOWA	7	—	1	8
BG	B.G.	—	—	7	7
UNK	UNKNOWN	6	—	123	129
	TOTAL:	3346	2930	4843	11127

SP3



CITY OF EDMONTON ENVIRONMENTAL SERVICES
WATER NETWORK MAINTENANCE TRAINING PROGRAM - OPERATIONS HANDBOOK

GENERAL SERVICES, PRINCIPLES & PRACTICES PAGE: 2-3-4
SERVICE QUALITY & CUSTOMER RELATIONS DATE: 19/01/89

Activity No.	Activity Name	Inventory	Quality Standard	Quantity Standard
121	Distribution Valve Operation	37453 Valves	Fully operating, 100% up, 100% down, no leaks	27% inventory/yr. (every 3.7 yrs.)
122	Chamber Cleaning	808 Chambers	Free of debris and water	82.5% inventory/yr. (cleaned every 1.2 yrs.)
123	Transmission Valve Operation/Greasing	808 Valves	Easily operate 100% open/closed	82.5% inventory/yr. (every 1.2 yrs.)
130	Hydrant Flushing/P.M.	11127 Hydrants	Clear water flows from hydrant; proper surface drainage occurs	100% inventory/yr. (every year)
131	Winter Checking Hydrants	11127 Hydrants	Dry barrel; hydrant operational	5 times/season
132	Hydrant Grounds Maintenance	11127 Hydrants	One metre of unobstructed access around hydrant	50% inventory/yr. (once every 2 yrs.)
133	Painting Hydrants	11127 Hydrants	Paint evenly distributed on a scale-free surface to slump point; meet color specification and numbering	22% inventory/yr. (painted every 4.5 yrs.)
140	Anode Excavating	1146 Magnesium Anodes	Main in center of excavation; sufficient workspace	6% inventory/yr.
141	Anode Excavation Shoring	1146 Magnesium Anodes	Meet Engineering approved standard	6% inventory/yr.

SP3

Customer Relations Development

Good customer relations are defined by the actions and information employees provide to give customers the satisfaction that service requests are handled with competence, concern and courtesy. Customer relations development is a goal this Water Network Maintenance training program helps achieve by incorporating two of the key components of customer relations into every course and workshop presented to trainees, i.e. service level knowledge and effective communications.

The linkage between these components is extremely important. An employee with thorough knowledge of all Water Network Maintenance service levels and the communication skills of a pitbull will most likely fail to convince a customer that the Section has the best interest of the customer at heart. Similarly, an employee with excellent communications skills but no understanding of service levels, will appear shallow, uninformed and leave a customer with the feeling that his/her service request or concern is being trivialized.

WORK ORIENTED GOALS

The day-to-day need for many Water Network employees to interact directly with the public or with specific customers, makes customers relations a high priority goal for the Section. Customer relations, though, is not a goal in itself for Water Network Maintenance. It is not the end result of professional accomplishment for employees, but rather the stimulus that reinforces a corporate climate in which Water Network employees relentlessly pursue service oriented work behaviour, work quality improvement, performance consistency improvement, and productivity improvement.

Service oriented work behaviour is defined as the extra care and effort an employee adds to the technical competence (s)he has to offer. Many of the Water Network activity methods described in the Operations Handbook of the training program feature steps that illustrate ways in which employees undertake extra effort to convince specific customers and the public of their services oriented behaviour through their actions. Actions which show that they are concerned equally with the safety, convenience and satisfaction of customers as they are with the condition and reliability of the Water Network infrastructure. Consider, for example, the service provided with the activity of 'Break Location Testing':

- Go to each premise and notify people of the water shutdown, if they have not yet been notified by a Troubleman:

  - Inform them when the shut off will take place.
  - Give the estimated time period of the shut off.
  - Leave a BLUE (Sorry for the Inconvenience) TAG, if no one is at the premise.
  - Call Dispatch regarding the shut off time and the reason, so that they can provide that information to (customers) calling about having no water.
  - Arrange for a temporary water hose hook-up for (customers) on the list labelled "Emergency Contacts When Water Off".

Work quality improvement is a goal which more specifically addresses the trade skills, craftsmanship and technical precision necessary to maintain repair and servicing dependability at the highest possible level and to reduce the chance of failure. Again, the activity methods in the training program's Operations Handbook, focus on this craftsmanship by offering an ideal vehicle for pride in work quality and satisfaction, this time for the employee (her)himself. Employees access the methods documentation to make improvement suggestions, and are directly involved in testing these improvement suggestions to obtain approval for implementation.

Performance consistency improvement is an ongoing goal which follows work and service quality. Service quality and customer relations are not considered to be good unless they have been maintained consistently at a high level over a longer period of time such as 3 to 5 years. To build this consistency, each employee must be aware of the responsibility (s)he has to improve public awareness and customer awareness of service quality by drawing attention to Water Network Maintenance's ability to maintain almost uninterrupted water service to approximately 3/4 million customers regardless of weather, temperature, or infrastructure conditions. The key contribution Water Network Maintenance training can give to this goal is to train each employee in using the best activity methods given various circumstances, to know why they are selecting that method, and to achieve consistency with their peers in applying the methods proven to be reliable and successful.

Productivity improvement is a goal which deserves significant attention once service quality, work quality and performance consistency are observed to be actively in progress. It can be defined as the amount of work output produced by a crew or an employee given a desired level of service quality, work quality and performance consistency. Productivity improvement is the result of methods improvement and successfully implemented technological development. Ideally such improvement is fuelled by employee initiatives based on the motivation to enhance the quality of their work time, their professional self-image, personal and career ambitions, and the status and respect bestowed by the public or by individual customers. Water Network

Maintenance training support this kind of productivity improvement and interest by incorporating these motivations into the courses and workshops, and by relating these motivations to successful employment at Water Network Maintenance.

EMPLOYEE ORIENTED GOALS

Job Interest Training

Employees with strong interest in job technology performed elsewhere in the organization are usually willing to commit effort to learning other jobs, even though no immediate openings are available. The benefit to Water Network in maximizing the number of employees knowledgeable in other work lies in strengthening in-house communications and cooperative work behaviour. Water Network has progressed into building an organizational climate aimed at good customer services reflecting courtesy, competence and concern. Widening the perspective of employees on the complexities and skill involved in other areas within Water Network helps build a unified presentation of service oriented behaviour to customers. Employees who are willing to invest time and effort in learning other jobs without up-front commitment will also make better candidates for employment into these areas should openings become available. New job learning time is reduced, temporary replacement [or backup] requirements are more easily fulfilled, and employee career development opportunities are being served.

Job Upgrading Training

New materials, equipment, tools, safety requirements and environmental concerns continually demand sensitivity to the development of employee skill and knowledge. The training program and the Training Officer play a key role in collecting, assimilating and timing the introduction of this progress into the workplace. Successful progress is made by identifying the work situations and best methods with a test group of employees, and by using the training process as a vehicle for further implementation through trainee involvement. Employee ability in applying new technology or procedures is a key accomplishment of the training program in this area. A faster and more effective way of bridging the gap between theory and practical applications is also part of this training accomplishment.

Job Transfer Training

Water Network's commitment to invest in well-trained, experienced and customer service oriented employees, is highlighted by the use of the training program as a starting point for employees to be transferred into different job responsibilities. The goal of job transfer training is to equip such employees with the knowledge and skill required to enter the workplace and become productive within a relatively short time period.

New Employee Training

New employees are first trained in the general services, principles and practices of the Water Network Maintenance Section. Then they receive specific job skill and knowledge training in the work activities for which they have been hired, with due emphasis on the goals of customer relations, service oriented work behaviour, work quality, performance consistency and productivity improvement.

COURSE/WORKSHOP COMPOSITION

Water Network Maintenance's goals are the starting point for course and workshop composition. The goals set forth what needs to be achieved by presenting the program, and course/workshop composition outlines which needs to be taught to what group of trainees within which guidelines.

Water Network Maintenance is a dynamic organization operating in an environment which often requires immediate response. Planned courses can handle the major training effort each year, but provision is made to accommodate on-demand workshops.

To give structure to a basic framework for composition the following definitions are provided:

Course A training program consisting of several consecutive and related one to three day sessions which:

a) produces a pre-qualified trainee ready to practice in the activities covered by the subject material (e.g. new employee becoming a Meter Installer), or which

b) conveys a new general organizational development goal (e.g. customer service orientation) to employees retraining in their own area of work responsibility.

Workshop A 1/2 to 3 day training program composed to satisfy a specific need due to technological change, organizational change or new legislative requirement (e.g. corrosion value testing workshop for Network Repair crews).

To gain maximum effectiveness from the training effort, the program includes a basic program composition procedures which the Training Officer follows to ensure success. A random response to training requests received from various supervisory staff members would render the training service ineffective, incomplete and unmanageable from a Training Coordinator standpoint.

For Water Network Maintenance training, the following composition procedure is used by the Training Officer:

- Investigate and assemble all training requirements for a new budget year, and sort them by training objective.

- Sort the requests by Water Network Maintenance area of responsibility, for employee oriented or work oriented goals:

  1) Water Network Distribution Preventive Maintenance & Testing

  2) Water Network Distribution Operations Repair/Replace

  3) Water Network Metering Operations

  4) Water Network Quality & Consumption Control

  5) Water Network Dispatch & Resource Services

- For employee oriented training sort the trainees by area of responsibility into trainee groups such as:

  Backup trainees - employees who will serve in other positions for vacation relief, etc.

  Refresher trainees - employees who need a better appreciation of all aspects of their current job responsibilities.

  Interest trainees - employees who wish to enroll in a program to learn another job in preparation for future openings or for other career development reasons.

  Transfer employees - employees who have been or are being transferred to a different position within Water Network Maintenance.

  New trainees - employees who have not worked for Water Network Maintenance before and are new to all general as well as specific procedures.

  Upgrade trainees - employees who need to learn how to use newly introduced technology, materials, or new procedures for safety, customer relations, etc.

- Select the basic composition framework to be used; i.e. is the request best handled as a course or as a workshop.

- Determine the training module to be delivered; is it based on a function or an activity? The minimum module for a course should be a function (a function is defined as group of work activities); for a workshop, an activity should be the smallest unit.

- Identify the minimum class size required for effective training. Generally a group of less than 5 becomes ineffective and unproductive from a training viewpoint. Combine trainee groups, if necessary, to reach the minimum class size, knowing that the course/workshop can be composed so that the needs of each trainee group is met. Crew sizes must be kept in mind, because many lessons are based on crew training methods, the class size must be a multiple of the crew size.

- Define the length of time required to deliver the course/workshop. Use the time ranges specified with each lesson plan in conjunction with the background of the trainee. Organize presentations in 3 hour time blocks.

- Prepare a course/workshop composition summary for approval by the Superintendent of Water Network Maintenance. This summary should include the following:

  - the name of the course/workshop
  - who requested the training
  - the names of the trainees
  - the functions and activity lessons to be taught
  - the primary objective to be achieved, its relationship to Water Network goals
  - the total training time in days
  - the trainers and guest lecturers required
  - the cost and the source of funding (which budget pays?)
  - the possible delivery dates and location(s)
  - request for authorization by Superintendent of Water Network Maintenance

- Complete this type of composition summary for each course/workshop required and project the likelihood of being able to re-use the same composition in the near future or the next budget year.

- Once approval is obtained, proceed to compose the course/workshop by selecting the required lesson plans from the Instructor's Handbook. Select from Section II as well as Section III. Review and adapt the training procedures outlined in the lesson plans to establish continuity throughout the course/workshop. Choose an appropriate delivery method [see: 'Course/Workshop Delivery' discussion]. Reflect on and incorporate where necessary Water Network's goals with respect to customer relations, service oriented work behaviour, work quality, performance consistency, and productivity improvement.

- Prepare topic tests and a course/workshop final exam by selecting the relevant questions from the bank of test questions in the Instructor's Handbook.

- Prepare overheads of forms and records from the Forms & Records Catalogue in the Operations Handbook. Prepare overheads of illustrations used in the Activity Methods in the Operations Handbook.

- Arrange for copies of the Operations Handbook for the trainees, or copies of the Activity Methods relevant to the course/workshop to be delivered.

COURSE/WORKSHOP SCHEDULING

Course/workshop scheduling must be sensitive to the constraints and opportunities inherent in the nature of Water Network Maintenance operations. When is the best time to offer training to whom, and under which conditions? Some courses are best scheduled for a calendar time when work pressures are generally known to be low. Good scheduling procedures will also give due consideration to daily start and finish times, as well as the number of hours of training per day needed to maintain maximum trainee participation and training effectiveness at the same time.

Since course/workshop composition process already determined the 'who, what and why' for the training program, scheduling should only address 'when and where'. There is no workload pattern common to all Water Network Maintenance areas of responsibility, so choosing one training session for the entire Section will be almost impossible. Scheduling will be the least difficult for a group of new employees hired from outside of Water Network Maintenance. All other trainee groups will impact the work environment in terms of workload pressure. Understanding the following general workload cycle for each area of responsibility in Water Network Maintenance helps to make the right scheduling decisions:

	WATER NETWORK AREA OF RESPONSIBILITY	HIGH WORKLOAD PERIOD	LOW WORKLOAD PERIOD
1.	Preventive Maintenance & Testing	Summer	Winter
2.	Operations Repair/Replacement	Winter	Summer
3.	Metering Operations	Winter	Summer
4.	Quality and Consumption Control	[No significant workload variations]	
5.	Dispatch and Resource Services	Spring/Fall	Winter

For courses it is important to schedule the training modules over a reasonable time frame. If, for example, the entire workforce is to be trained for greater performance consistency, then such a program objective is given a distinct time period during which it is to be completed. Similarly, a course given to new employees should have a limited time frame. There is little or no reason to exceed a one year time frame for any course at Water Network Maintenance. As indicated in the course/workshop composition discussion, a course is planned and scheduled in segments of 1 to 3 days. Workshops may be as short as one half day, but not longer than 3 days.

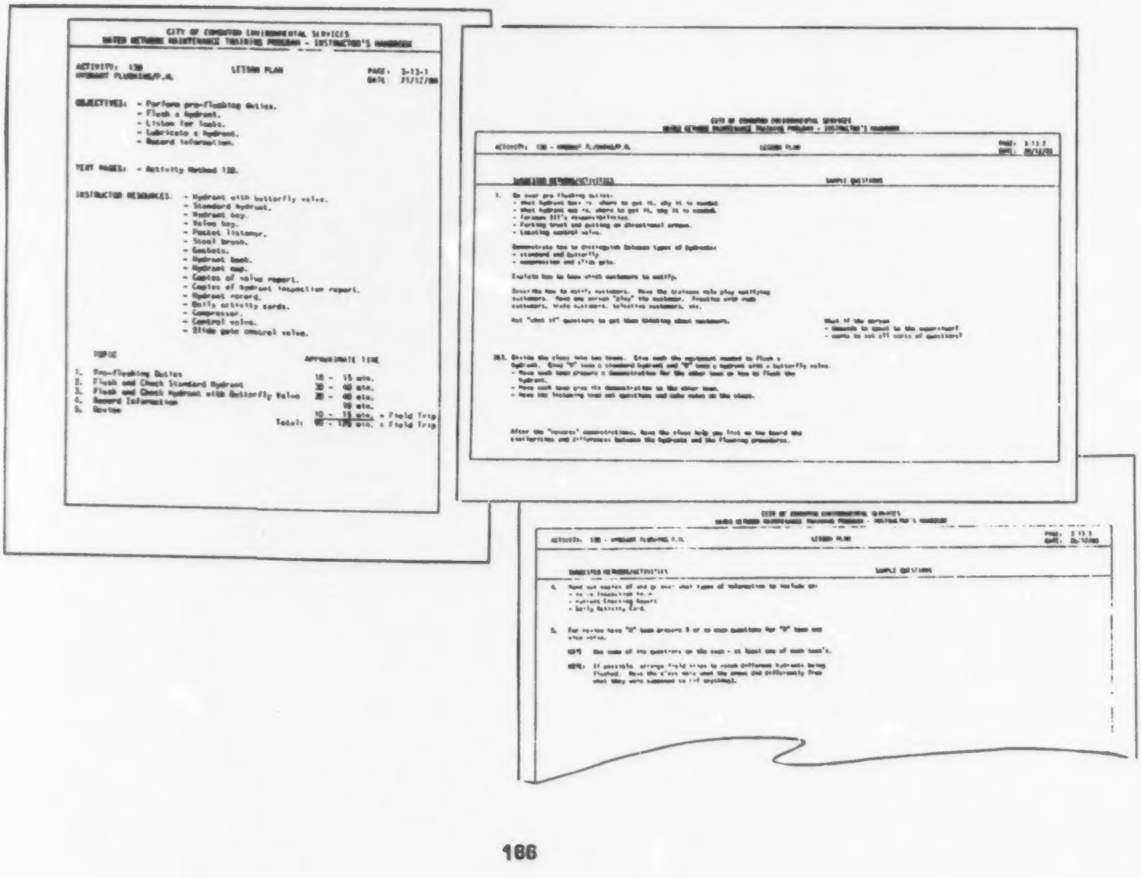
Training days run from nine to four, basically offering two 3 hour sessions per day. Exceptions are made if field trips or shop experiments are planned.

COURSE/WORKSHOP DELIVERY

There are three key aspects to the training delivery approach set up for the Water Network Maintenance Training Program:

- Tell-Show-Do-Review method of training
- Crew oriented training, and
- Position oriented training

The Training Officer of trained in the basics of job training and adult learning techniques, and the lesson plans in the Water Network Maintenance Instructor's Handbook (see illustration), are designed to guide the Trainer through the training process with consistency and professional integrity.



The 'Tell-Show-Do-Review' method is as simple as it sounds. The Trainer first explains ('Tell') the learning objectives and the subject matter; then (s)he demonstrates ('Show') how the work is done, or illustrates a procedure by using maps, charts or graphs. The third step is to have the trainee practice ('Do') under supervision, sometimes with the help of peers. The last step is to review the lesson with the trainees to reaffirm their understanding of the subject matter.

Crew oriented training is designed for those activities performed by crews. The importance of crew training should not be under estimated. Water Network trainees not only need skill and knowledge development, but also teamwork training. The lesson plans accomplish that, so that when trainees are at the work site they are aware of the responsibilities of the entire crew and know their role in achieving the desired teamwork results.

Position oriented training focusses on individual jobs which are not dependent on crew member cooperation, e.g. Meter Installers, Customer Service Agents, Dispatchers. The lessons plans for these positions cover in-class practice sessions that strengthen the need for self-reliance and self-correcting work behaviour. Careful reporting of information on forms and records is stressed in these sessions.

In general the delivery techniques incorporated into the lesson plans emphasize trainee participation in discussion, role playing exercises, and practice with props (a hydrant, a valve, meters, shoring, pipe, etc.). Other training techniques such as case studies or lectures are not used formally or are used only minimally. Case studies can be introduced informally by the Training Officer based on his experience with the subject matter under discussions, and in some cases the role play scenarios in the lessons can be transformed into case studies at the discretion of the Training Officer. More extensive use of the lecture approach is, however, not recommended.

Good lesson plans and training techniques should be delivered in a suitable environment. Every possible effort is made to deliver courses and workshops in a well-equipped training room, with sample space to demonstrate the use of props and audio-visual materials. Trainees are seated at chairs and tables to review training materials such as the activity methods, forms and records. They have sufficient workspace in the classroom to write tests and exams. The objective is to arrange a comfortable classroom environment without physical discomforts which might become obstacles to the learning process.

At present the Water Network Maintenance training program does not accommodate tutor assisted self-learning procedures, computerized or manual. It should be noted, however, that the Section's experience within the next three to five years will open up excellent opportunities to move into that direction. Keeping the program up-to-date with the changes in the workplace, and keeping the documentation of activity methods and lesson plans current, will be key factors in determining the feasibility of such developments.

COURSE/WORKSHOP EVALUATION AND FOLLOW-UP

The Instructor's Handbook includes more than 150 pages of test questions and answers. The questions are used by the Training Officer to compile a unique set of questions for each course or workshop every time these courses/workshops are delivered. The answers are provided to guide the Training Officer in maintaining consistency in evaluating and marking the test results from trainees.

Trainee Evaluation

The basic framework recommended for testing is one test for every 3 hour training session (1/2 a training day), and an exam at the end of a program. Exams are administered and controlled by the Training Officer with strict instructions to trainees not to provide assistance to others in answering the questions. The exams are marked by the Training Officer who delivered the course or workshop. Marks are recorded in % points based on the number of questions answered successfully. Questions can be weighted at the discretion of the Training Officer, given that the weight values are openly printed with the questions on the exam papers given to the trainees. Exam results are kept confidential and released only to the trainees themselves. Exam papers are retained by the Training Officer and filed by employee name. Discussions with trainees regarding exam results are arranged by appointment at the Training Officer's office. At such discussions the trainee may receive the Training Officer's opinions on the reasons why (s)he has done better or worse than expected.

Water Network Maintenance can establish a minimum passing grade for all courses and workshops. For most of the course and workshops 70-75% will be a reasonable minimum grade. A higher minimum can be used if the course/workshop deals with public safety or worker safety training.

Trainee evaluation and progress is difficult to manage without trainee records. Hence it has been recommended that a separate file be maintained for every trainee, and that such a file contain the following items:

- trainee name
- employee number
- training interests and/or requirements
- Water Network Maintenance course(s) taken:
  - attendance record
  - test results and papers
  - exam results and papers
- Water Network Maintenance workshop(s) taken:
  - attendance record
  - test results and papers
  - exam results and papers
- Coach assigned (see discussion on 'Evaluation Follow Up')

A summary record by employee of these trainee records will be compiled by the Training Officer for review by the Water Network Superintendent. If desired this record can be shared with the Departmental Human Resource Coordinator for official recognition of career development accomplishment.

Instructional Evaluation

At the conclusion of each course or workshop, trainees should be asked to evaluate the program as they experienced it. This evaluation should be handled anonymously and be in a written format. The evaluation form is basically established by the Training Officer and approved by the Water Network Superintendent; the following questions are included:

COURSE/WORKSHOP MATERIALS:

	USELESS				ESSENTIAL
1. How useful were the handouts?	1	3	5	7	9
2. Were the props adequate?	1	3	5	7	9
3. What feedback did test results give you about your progress?	1	3	5	7	9
4. What improvements would you suggest to the course materials?					

INSTRUCTION:

	USELESS				ESSENTIAL
1. Were objectives clear to you?	1	3	5	7	9
2. What do you think of the learning exercises?	1	3	5	7	9
3. What do you think of the field trips or shop sessions?	1	3	5	7	9
4. How useful was the instruction?	1	3	5	7	9
5. What improvements would you recommend to the instruction process?					

TRAINEE:

1. Are you attending the course as a result of your own interest and initiative?	YES / NO
2. Did you attend all the instruction time?	YES / NO
3. Did you miss instruction time because you were called back at work?	YES / NO
4. Are you going to recommend this course/workshop to others?	YES / NO

Evaluation Follow-up

Operations training programs are useful only if trainees can apply what they have learned and receive support from their Foreman or Supervisor in doing so. Achieving these two objectives requires a follow-up plan beyond classroom training. Assigning a "Coach" to each trainee is considered to be one of the best strategies in getting Foremen, Supervisors or Leadhands to share responsibility in successful training. The role of the Coach is to assist a trainee with:

- answering questions about procedures or methods not yet fully appreciated by the trainee.
- correcting improper actions that could undermine safety, customer service, work quality, or cost effectiveness.
- teaming him/her up with a highly experienced employee who has a suitable personality to be patient with people who are learning.

The role of the Coach is also invaluable to the Training Officer, and the training program, in that:

- the firsthand experience with trainees puts a Coach in a position where (s)he can provide feedback to the Training Officer to strengthen the training program where necessary.
- the Coach can advise the training officer when a trainee is considered to be fully trained to play an equal role in a crew setting, or to perform independently in a position.

Although the Coach can be assigned to a trainee at the beginning of a course, an agreement is made to end it within 3 months after the course exam. Both the Coach's comments and the trainee's are included in the trainee file when that 3 month period is finished.

Once a year the Training Officer is advised to conduct a half day discussion period with all those who have functioned as a Coach in the previous 12 months. The purpose of this session is to evaluate the coaching concept and gain an appreciation of how staff want to see it changed to improve its effectiveness.

TRAINING PROGRAM MATERIAL MAINTENANCE

The Training Officer has been assigned the responsibility to revise, modify, update or otherwise develop the documentation associated with the Water Network Maintenance Training Program. The update procedures are included in each handbook, and new materials is incorporated in a compatible format. One of the key issues is maintaining strict security over the lesson plans and the test question/answer bank. Circulation of updates to the Instructor's Handbook is limited to the Training Officer. The test questions and answers are stored off-line on a computer diskette and secured under lock and key.

Finally, at the end, I would like to comment as to how we are using this program since it was completed.

Out utmost interest is to have all employees fully trained. However, it is impossible to obtain this goal in a short time.

To increase the use of the Training Manuals and that the training become a way of life (work life) we split the training in two functions.

One is training by Training Officer, second is training by Foremen.

All new employees are trained by the Training Officer at their starting date. Training consists of safety requirements, familiarization with departmental and water branch organization and general understanding of the water production process from water treatment to water distribution and maintenance activities.

Training by the Foremen is done by the following process. Each month Supervisors select a new procedure that is applicable to the seasonal work. This procedure is presented to Foremen III's by the Training Officer and he shows them how to present and train the next level of Foremen (I's). Foremen I's then train their crews. This process helps to bring the entire organization on the same level of knowledge and methodology of work. It also ensure that the newly completed training manuals do not start accumulating dust. To find out the retention level, each session ends in short quiz or exam. Some time the new method or improvements to the present method are suggested by employees and these are incorporated into the procedures by the Training Officer.

The training program, by continuously involving the employees in incorporating new operation developments, results in improvements in productivity and customer relations.

This, again, ensure our organization doesn't become static, but is progressive, flexible and very much alive.

RESEARCH INTO THE DEVELOPMENT OF A BIOLOGICAL ACTIVATED SEWAGE TREATMENT SYSTEM

Joseph F. Salann, M.Sc.
Senior Environmental Consultant
David Bromley Engineering (1983) Ltd.
Edmonton, Alberta

INTRODUCTION

Sewage treatment for small communities, recreation areas such as camp grounds, ski facilities, resorts, and for certain types of industrial applications, is an unreliable, troublesome and expensive venture when one considers treatment on a cost per unit volume basis. In the past few years, many of these locations have been looking for alternatives to the conventional activated sludge facility or the facultative sewage lagoon approach.

These systems have been used traditionally in most small communities or recreation areas because they are relatively simple to operate and easy to maintain.

The problems associated with mechanical treatment systems or lagoons are as follows:

1. The start up time of an activated sludge system, whether extended aeration or RBC type, is too long for locations that generate a large portion of the wastewater in a short period of time. These are primarily recreation types of facilities such as a ski hill or camp ground where up to 90 percent of the sewage load could be generated within two or three days of the week. By the time the system produces acceptable water quality the load period is over and the system is shut down.

2. The construction of sewage lagoons require a significant amount of land. In areas where space or soil conditions are a problem or extremes in weather are considered as constraints, these types of treatment alternatives are difficult to utilize. Anaerobic treatment processes have typically been used for the treatment of high strength organic wastewaters and sludges. Recently, these processes have been considered for the treatment of lower strength municipal sewage. Resistance to the use of anaerobic processes for such is based on the high percentage of systems that failed to provide the required effluent quality. Anaerobic bacteria, which are the key organisms in anaerobic digestion, are slow to develop and are sensitive to temperature and pH variations in their environment. Environmental problems associated with the operation of these systems are also significant, particularly in a park environment where foul air emission is a concern.

Because of the above outlined problems, a number of research projects have been initiated with a view to developing more reliable and environmentally acceptable treatment alternatives. With the assistance and funding of Alberta Environmental Research Trust Fund, David Bromley Engineering (1983) Limited (DBEL) began a research project in 1988. The purpose of the research was to develop an alternative treatment process that would meet the following requirements:

1. Process reliability.
2. Ease of operation and simplicity of maintenance.
3. Ability to treat fluctuating and intermittent waste load.
4. Insensitivity to toxic influents.
5. Suitability for cold climate operation.
6. Rapid start up time.

Based on the above objectives, a biological carbon adsorption system was selected for the research program.

Recent water and wastewater treatment research has shown promise for fixed film biological reactors. Various materials, including sand, coal, soil and synthetic materials have been used for biogrowth support media in these systems.

For the DBEL research project, activated carbon was selected as the support media because of the very high surface area per unit volume of this material.

PROCESS DESCRIPTION

Traditionally, physical/chemical types of plants have used activated carbon in the following two applications:

1. To upgrade the effluent from the biological treatment plant.

2. To replace a conventional biological treatment system.

Carbon removes organics from wastewater through the action of two distinctively different mechanisms. The first of these is adsorption which actually removes the dissolved organics from the influent by intermolecular forces on the surface of the carbon granules where these organisms become substrates for biological activity.

It is theorized that adsorption is the principle mechanism by which dissolved organics are removed from the sewage. In this case, biological activity functions as a continuous regeneration process of the adsorption sites by degrading the organic materials and reopening the porous surface of the carbon. The exact mechanism is unknown but the pilot plant data has shown that the biological contribution to the removal process is quite significant. Adsorption is probably predominant when a carbon is first placed in service. As operation proceeds, the importance of the biological process increases as the quality of biofilm around the carbon particles increases.

LABORATORY SCALE RESEARCH PROGRAM

Prior to initiating a pilot scale project, a laboratory scale program was undertaken. The purpose of this phase of the research was to develop some preliminary information regarding the operation of the carbon system and a design basis for the pilot plant. The laboratory work was performed on batch tests using clarified sewage from the Devon Wastewater Treatment Plant. The feed to the carbon columns was prepared by adding a primary coagulant (Ferric Chloride) and a coagulant aid. The separation of the solids took place in approximately 20-30 minutes. The biological oxygen demand load of the feed to the columns was approximately 38 mg/l. The adsorption columns were 25 mm in diameter and contained a 1 m deep carbon bed. The mode of operation was gravity-downflow. All tests were performed under approximately the same operating conditions with a flow rate of 20 ml per minute which is equivalent to a volumetric loading of 60 m^3/m^3 min.

LABORATORY SCALE OBSERVATIONS

Figures 1 and 2 show the type of BOD removal profile achieved using fresh activated carbon at 14° and $25^{\circ}C$.

FIGURE 1

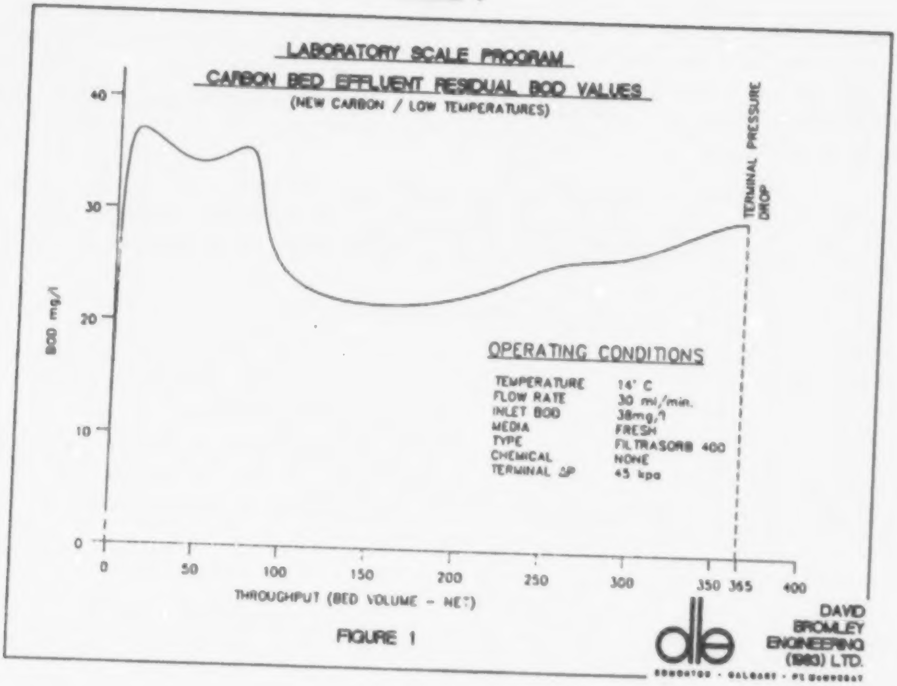
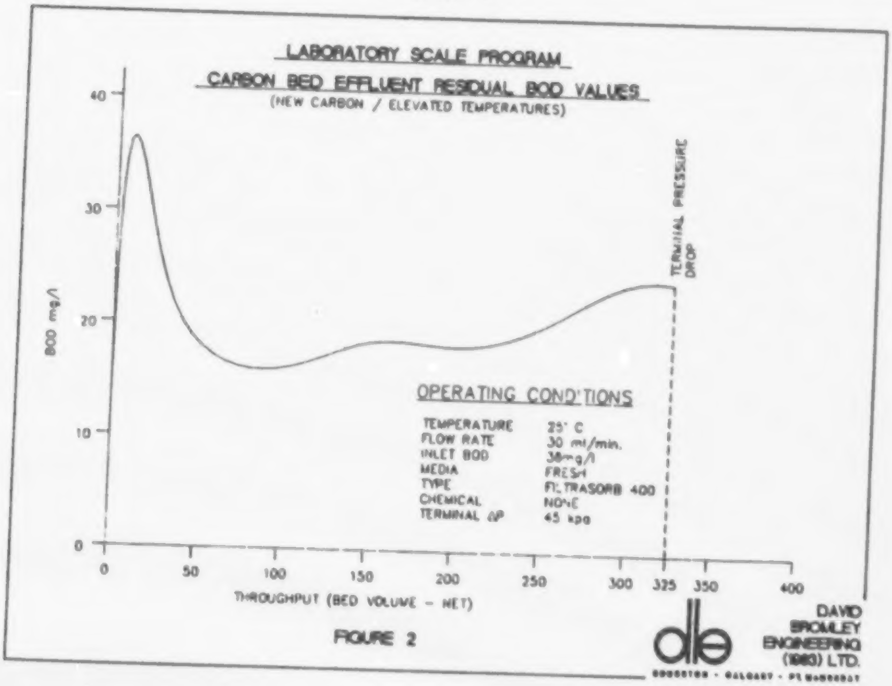


FIGURE 2



As illustrated by the graphs, BOD removal is almost 100 percent in the early stages of the run. However, within a short period of time [after approximately 4 to 5 bed volumes], BOD removal diminishes to almost zero and the effluent BOD is about the same as the influent. As the run progressed, the effluent quality improved. This is most likely due to the development of a biofilm on the surfaces of the carbon particles and the resulting biodegradation of organic matter present in the sewage.

The above observations are supported by the fact that the BOD removal is a function of temperature. The best removal at 14°C was approximately 16 to 18 mg/l while at 25°C it was in the range of 22 to 24 mg/l. The adsorption process is not affected by temperature variations.

Following the column tests using fresh carbon, experiments were undertaken using old carbon material. Results are shown in Figures 3 and 4. The BOD removal profile of these tests are quite different from the fresh carbon bed profiles. The characteristic high removal efficiency associated with the early stages of the run are absent. The very low BOD removal rates during the early stages of the run with the old carbon material represents a segment where neither adsorption nor biological degradation is taking place. However, as biomass on the carbon particles increased, effluent quality also improved. As indicated by Figures 3 and 4, the BOD removal efficiency is again influenced by the operating temperature of the system.

FIGURE 3

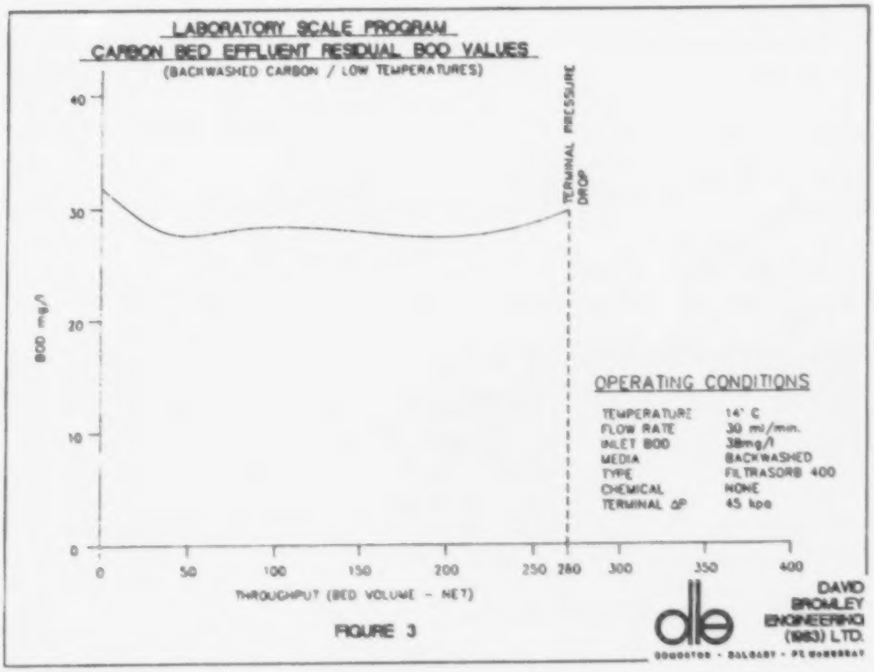


FIGURE 4

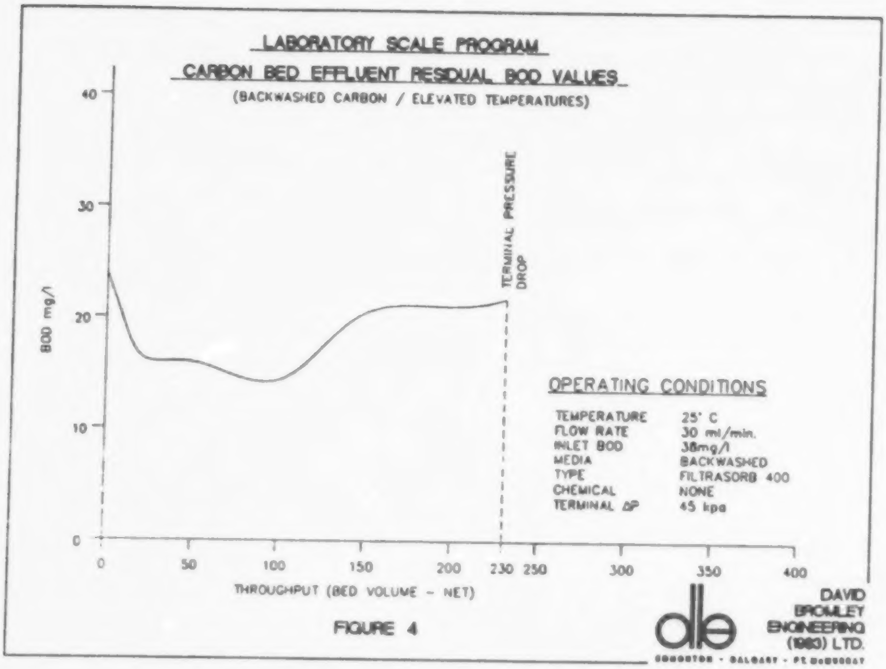


FIGURE 4

During the laboratory scale columns tests, it was noted that the system went septic rapidly and hydrogen sulphide production could be observed. In addition, a black mat started to grow on the surface of the carbon bed. This mat was most likely responsible for the high pressure drop across the column. The effect of temperature on the rate at which this mat was formed can be observed on the BOD profiles. The terminal pressure drop of 45 kPa was achieved at a lower volumetric throughput at 25^0 than at 14^0C.

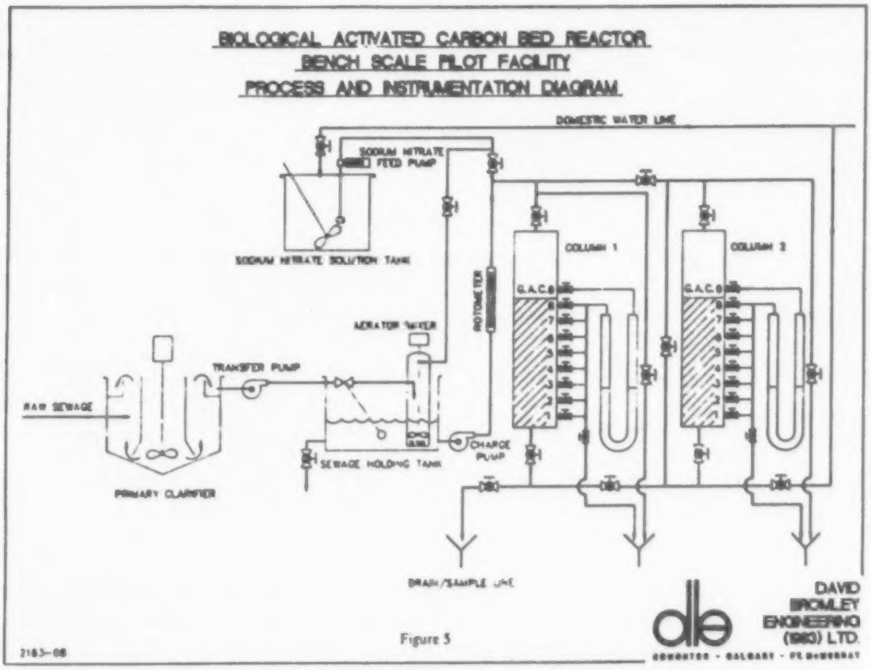
An analysis of the hydrogen sulphide production problem indicated the establishment of an abundant anaerobic bacterial growth. This growth was further accelerated as a result of high operating temperatures. Our solution to the problem was the addition of sodium nitrate ($NaNO_3$) to the carbon column influent in concentrations of approximately 10 to 25 mg/l.

It was felt that the problem was caused by the utilization of sulphate by the bacteria. The anaerobic bacteria split oxygen from the sulphate molecule for their metabolism. By-products of this reaction are hydrogen sulphide and metallic sulphides which show up in the effluent as a black precipitate as well as the matting on the surface of the carbon bed. However, in the presence of $NaNO_3$, the bacteria will preferentially metabolize the nitrate rather than the sulphate to obtain oxygen. The by-products in this case are nitrogen gas and carbon dioxide, neither of which are objectionable. In addition, the dense matting which interferes with carbon column flow rates is also greatly reduced or completely eliminated. As a further benefit, BOD removal efficiency of the columns had also increased since the bacteria uses soluble carbon from the column influent for cell growth. Apparently, the bacteria enter into an endogenous respiration phase during some periods so that there is no gross sloughing of bacteria cells.

PILOT SCALE EXPERIMENTAL PROGRAM

Based on the results of the laboratory scale research, a pilot plant was constructed and placed into operation at the Devon Wastewater Treatment Plant. A process and instrumentation diagram of the system is shown in Figure 5.

FIGURE 5


BIOLOGICAL ACTIVATED CARBON BED REACTOR
BENCH SCALE PILOT FACILITY
PROCESS AND INSTRUMENTATION DIAGRAM
DOMESTIC WATER LINE
SODIUM NITRATE FEED PUMP
SODIUM NITRATE SOLUTION TANK
COLUMN 1
COLUMN 2
G.A.C.
ROTOMETER
AERATOR WATER
TRANSFER PUMP
RAW SEWAGE
SEWAGE HOLDING TANK
CHARGE PUMP
PRIMARY CLARIFIER
DRAIN/SAMPLE LINE
DAVID BROMLEY ENGINEERING (1983) LTD.
EDMONTON - CALGARY - FT. McMURRAY
Figure 5
2163-08

The operating and monitoring procedures established during the laboratory work were used with the following three major deviations:

1. The power plant was operated at ambient temperatures ranging from 11° to 15°C.

2. The influent to the pilot plant was sewage from one of the primary clarifiers at the wastewater treatment plant. This clarifier operates without chemical additions. As a result, the pilot plant influent BOD was higher than that of the laboratory columns.

3. The headloss profile across the columns were measured at 150 mm intervals using a mercury manometer.

A total of 24 experiments were performed during the 1988 pilot plant program. These experiments were performed with a view to the following criteria.

1. Demonstrate the effect of sodium nitrate addition on:

   a) BOD removal efficiency.
   b) System performance with respect to headloss across the columns and throughput capacity.

2. Determine the relationship between influent and effluent DO levels and BOD removal efficiency.

3. Determine the relationship between loading rates and BOD removal efficiency.

4. Provide preliminary information regarding system startup/shutdown and performance.

Based on the above objectives an experimental program was designed and undertaken.

PILOT PLANT OBSERVATIONS

Group 1 (experimental runs 1 through 8)

The purpose of these tests was to gain experience with the operation of the system and to determine how the equipment responds to the controlled variables.

No chemical additions were made during the first three tests. Typical test results are shown in Table B-1 and Figure B-1.

TABLE B-1

Experiment No: 1

Date Start: June 1, 1988 Finish: June 3, 1988
Total Time in Operation: 15 hours

Operating Conditions:

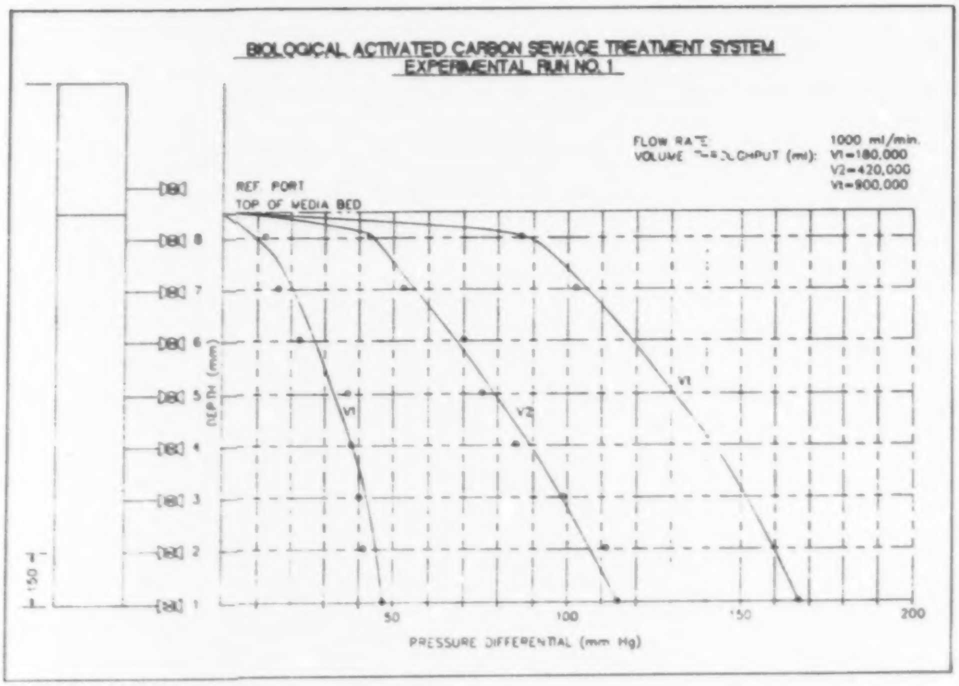
Carbon Bed Volume: 10.077 l
Flow Rate: 1000 ml/min.
Storage Tank Retention Time: 112.5 min.
Column Retention Time: 10.1 min. (4.6 min. net)
Sodium Nitrate Feed Rate: 0

Test Results

Sample No:	Influent Sample	V1 3 hrs	V2 7 hrs	V3 10 hrs	V4	Terminal Sample
BOD (mg/l)	96	87	78	66		71
% Reductn		9.5	18.8	31.0		26.0
Turbidity (NTU)	77.5					8.5
% Reductn						89
Throughput (ml)		180,000	420,000	600,000		900,000
Effluent O_2						
Effluent H_2S						
PressLoss (mmHg)		46	115			177

FIGURE B-1

Pressure Differential Curves



As indicated by the data, the BOD removal efficiency was in the range of 30 percent and the turbidity reduction was about 90 percent. The pressure differential curves show that most of the headloss takes place in the top 75 mm of the bed and utilization of the lower layers appears to be minimal. The addition of sodium nitrate to the feed had a marginal effect on effluent quality and system performance. Hydrogen sulphide production was reduced, the surface blinding of the media was eliminated and BOD removal efficiency increased by 15 to 20 percent.

Group 2 (experimental runs 7 through 12)

The purpose of these tests was to investigate aerobic and anaerobic operating conditions. Typical results are shown in Table B-2 and Figure B-2.

According to these tests, maintaining a small amount of dissolved oxygen was necessary to improve BOD removal efficiency. If dissolved oxygen was present in the effluent, the BOD removal efficiency of the system could be increased to around 37 to 38 percent. The amount of dissolved oxygen did not appear to be a critical factor as long as a trace was present.

The port pressure differential curves also indicate a substantially improved bed development profile. These tests did not indicate the characteristic large headloss at the surface of the carbon bed. While biological growth appeared to take place to approximately 0.5, which is the bed centre, as the run progressed additional biomass development could be observed in the lower portions of the carbon column.

TABLE D-2

Experiment No: 2

Date Start: June 1, 1988 Finish: June 3, 1988
Total Time in Operation: 20 hours

Operating Conditions:

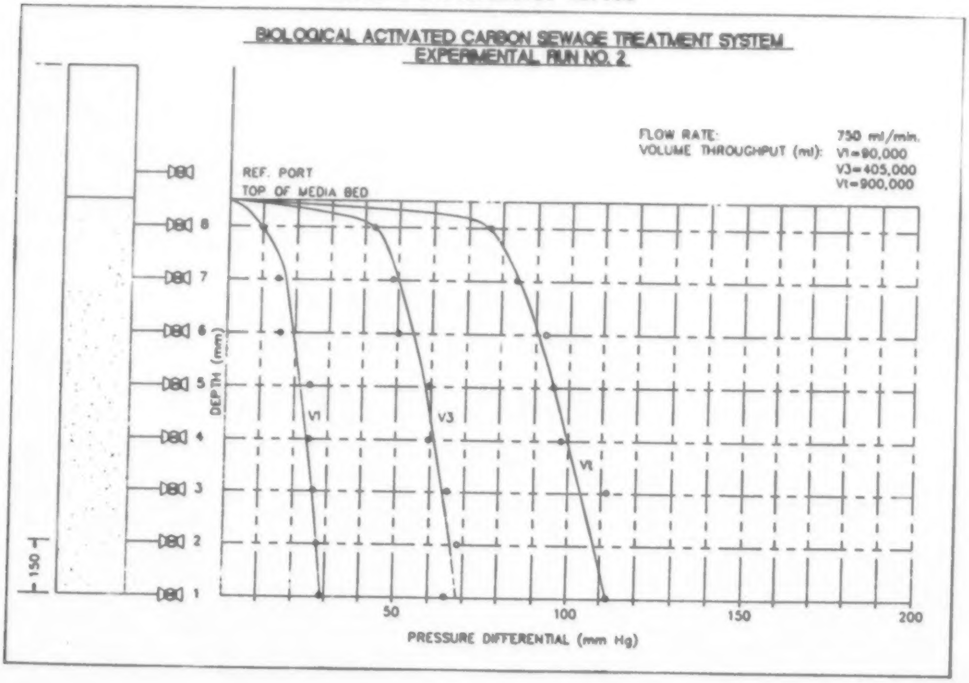
Carbon Bed Volume: 10.077 l
Flow Rate: 750 ml/min.
Storage Tank Retention Time: 150 min.
Column Retention Time: 13.5 min. (6.1 min. net)
Sodium Nitrate Feed Rate: 0

Test Results

Sample No:	Influent Sample	V1 2 hrs	V2 6 hrs	V3 9 hrs	V4 14 hrs	Terminal Sample
BOD (mg/l)	96	898	67	66	61	67
% Reductn		6.5	28.5	31	36	30
Turbidity (NTU)	77.5					7.0
% Reductn						91
Throughput (ml)		90,000	270,000	405,000	630,000	900,000
Effluent 0_2						
Effluent H_2S						
PressLoss (mmHg)		28		66		113

FIGURE D-2

Pressure Differential Curves



Group 3 (experimental runs 13 through 18)

These tests were designed to evaluate the effects of loading rate and retention time variations on system performance. Typical results are shown in Table B-3 and Figure B-3. These results indicate the performance of the two carbon columns in series. This mode of operation allowed the retention time to be increased by 100 percent and the BOD removal efficiency increased to 80 percent. The turbidity reduction was in the 90 percent range and with the addition of 13 mg/l of sodium nitrate, hydrogen sulphide production was minimal.

As is shown in the port pressure differential curves, an almost uniform bed development profile could be achieved. The high pressure loss across the top 50-75 mm segment of the bed was eliminated and a linear resistance to flow throughout the bed was observed. Visual observations of the carbon bed during this group of experiments indicated the growth of dark to medium brown biomass that could be distinguished from the black carbon particles. This was in contrast to the black matting that was noted when the system was operated under conditions that resulted in high levels of hydrogen sulphide in the effluent.

TABLE B-3

Experiment No: 16

Date Start: July 18, 1988 Finish: July 21, 1988
Total Time in Operation: 24 hours

Operating Conditions: Two columns operating in series

Carbon Bed Volume:	21.54 l
Flow Rate:	500 ml/min.
Storage Tank Retention Time:	8 min.
Column Retention Time:	40.4 min. (18.18 min. net)
Sodium Nitrate Feed Rate:	15 mg/l

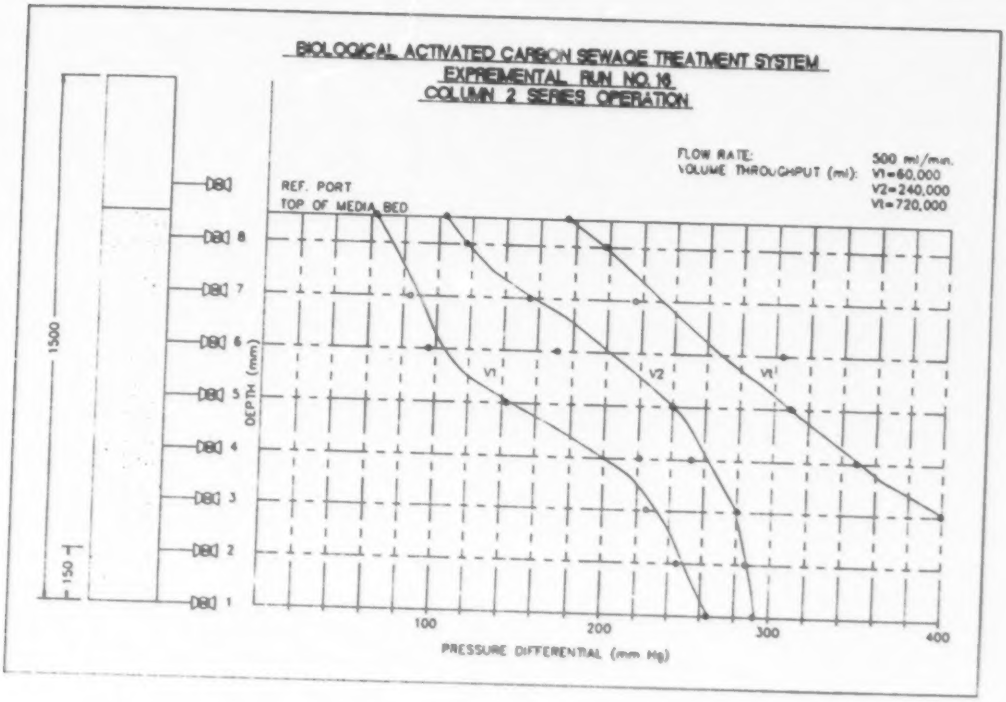
Test Results

Sample No:	Influent Sample	V1 2 hours	V2 8 hours	V3 16 hrs	V4	Terminal Sample
BOD (mg/l)	93.0	34	28	27.5		36.5
% Reductn		63.5	70.0	70.5		61
Turbidity (NTU)	48	3.4	4.8	26.5		7.5
% Reductn		93	90	45		85
Throughput (ml)		60,000	240,000	480,000		720,000
Effluent O_2		0	0	0		0
Effluent H_2S		0	Trace	Trace		Trace
PressLoss (mmHg)		265	293	310		485

Notes: Pressure loss values represent total headloss across two columns operating in series.

FIGURE B-3

Pressure Differential Curves



Group 4 (experimental runs 19 through 24)

The purpose of this group of experiments was to evaluate the effects of carbon column backwashing on system performance and to determine the length of time required to regain pre-backwash BOD removal efficiency. Typical results are shown in Table B-4. It may be noted that approximately 45 - 65 percent of BOD removal efficiency is lost due to a five minute clean-up operation and up to 90 percent of the BOD removal may be lost as a result of a 10 minute backwash operation.

It is also apparent that after approximately 45 minutes to 1 hour of restart, up to 75 percent of the lost efficiency could be regained.

Table B-4

Experiment No: 22

Date Start: August 11, 1988 Finish: August 11, 1988
Total Time in Operation: 6 hours

Operating Conditions: Two columns operating in series

Carbon Bed Volume:	21.54 l
Flow Rate:	550 ml/min.
Storage Tank Retention Time:	7.3 min.
Column Retention Time:	36.7 min. (16.5 min. net)
Sodium Nitrate Feed Rate:	12 mg/l

Test Results

Sample No:	Influent Sample	V1 0.5 hours	V2 2.5 hours	V3 2.95 hrs	V4 5.55 hrs	Terminal Sample
BOD (mg/l)	73	38	34.5	50	21	46
% Reductn		48	53	32	71	37
Turbidity (NTU)	62	4.6	7.1	6.2	4.3	5.6
% Reductn		92.5	88.5	90	93	91
Throughput (ml)		16,500	82,500	97,350	183,150	198,000
Effluent 0_2						
Effluent H_2S						
PressLoss (mmHg)						

Notes: Both columns backwashed at V_2 & V_4 for 3 minutes. Samples V_3 and V_4 taken 20 minutes after return to service.

PILOT PLANT EXPERIMENTS - DISCUSSION OF RESULTS

The pilot plant study demonstrated the interaction of surface loading rate, dissolved oxygen levels, and sodium nitrate feed rates.

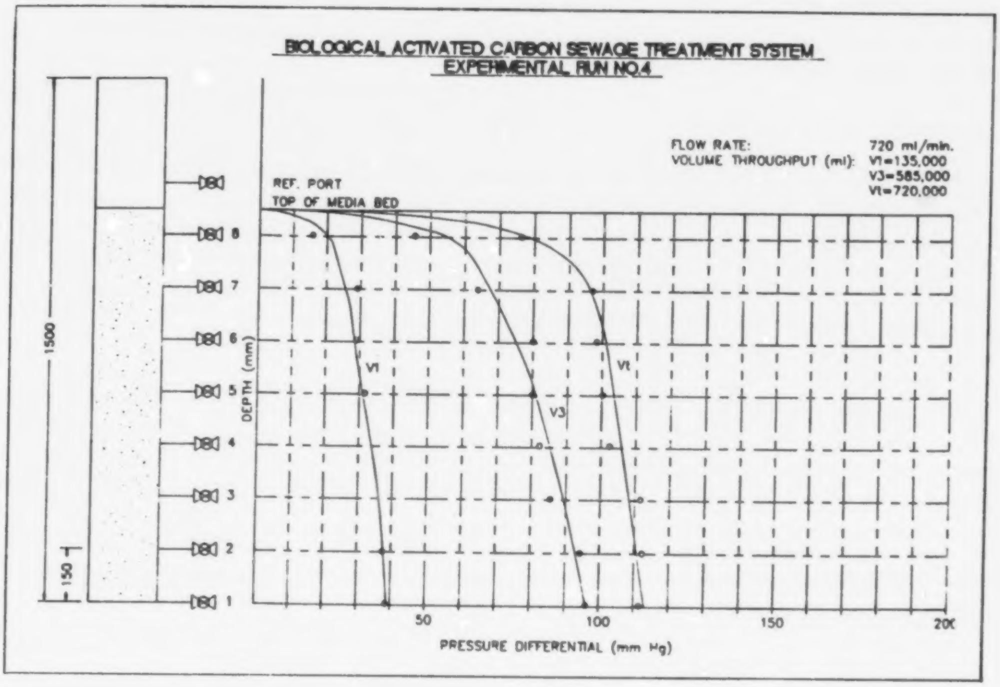
The effect of these three variables on system performance are as follows:

1. Surface loading rate. The flow rate through the carbon bed and the resulting hydraulic retention time or residence time within the carbon columns demonstrated a direct effect on BOD removal efficiency. Decreasing the flow rate improved BOD removal. Retention time appeared to operate independent of other factors indicating that within a critical range, BOD reduction may be improved by reducing the hydraulic loading on the system. The surface loading rate also appeared to influence the geometry of the biomass development in the reactor. Increasing flow rates tend to promote a more linear bed development. As shown in Figures B-1 and B-4, the high flow rate systems, (operating at 1000 ml per minute), at the conclusion of the run approximately 60 percent of the total headloss occurred in the top 225

mm. When the system was operated a lower flow rate (720 mm per minute) 90 percent of the total headloss occurred in this same region.

FIGURE B-4

Differential Pressure Curves



2. Dissolved oxygen levels. The results of the experimental work show that the highest level of BOD removal would be obtained when at least a trace of dissolved oxygen could be detected in the column effluent water. The presence of dissolved oxygen in the effluent is determined by a number of interactive variables. Some of these variables are:

A) Dissolved oxygen in the influent.
B) Residence time in the settled sewage storage tank.
C) Residence time in the carbon column.

In addition to the above, biological activity in the storage tank and in the carbon bed will also effect effluent DO values. This biological activity in turn is influence by a number of uncontrolled variables such as temperature, BOD loading, etc.

The data indicates that the level of DO in the effluent does not appear to influence the BOD removal efficiency. As long as a trace of oxygen is present, the system will produce 40-50 percent BOD reduction in a single column and 60-70 percent reduction when two columns are operated in series. With zero dissolved oxygen in the effluent, and all other variables held constant, the BOD removal was in the range of 15-25 percent.

3. Sodium nitrate feed rates. The addition of sodium nitrate to the columns influent was investigated under both aerobic and anaerobic conditions. The addition of sodium nitrate under anaerobic conditions had minimal effect on the system performance. However, when the system is operated under aerobic conditions, the addition of sodium nitrate, in the range of 5-15 mg per litre, substantially improved system performance.

The effects of sodium nitrate feed are noted in two areas:

1. Hydrogen sulphide production is reduced.

2. Bed development geometry is improved.

It was noted that during initial stages of an experimental run, oxygen would be adsorbed by the carbon. However, as the run progressed, the adsorption would have less and less significance. During these early stages of the run, the addition of sodium nitrate appeared to have the effect of reducing or completely eliminating hydrogen sulphide production by the anaerobic biological activity. It is hypothesized that localized anaerobic conditions would develop in the carbon bed even though the system basically operates as an aerobic system. Sodium nitrate feed was beneficial in eliminating these localized conditions.

The addition of sodium nitrate also altered the physical characteristics of the biomass and the resulting bed geometry development. Without sodium nitrate addition, surface blinding was much more likely to take place. The biomass had a black appearance and it formed a mat on the carbon surface which has a high resistance to flow. This black mat tended to build on the interval pressure ports, thus obstructing headloss profile analysis.

The runs where sodium nitrate addition was used produced a more granular type of biomass which was not restricted to the surface layers. The headloss profiles for these runs show a much more linear bed development indicating biomass growth throughout the media.

The effect of the above outlined variables is complex. Influent DO levels, surface loading rates and chemical feed rates interact to affect the BOD removal efficiency of the system as well as the length of the service cycle. Hence, there is an optimum balance of these variables.

The data seems to indicate that there are divergent trends for the following variables:

1. Surface loading rates which are necessary for uniform biomass development.

2. Column retention time required for efficient BOD removal.

This is because all parameters based on these variables may pass through optimum values rather than producing a single positive or negative response.

With respect to suspended solids reductions, the system operated efficiently. Operating variables outlined above did not appear to effect suspended solids removal. Suspended solids reduction in excess of 90 percent could be demonstrated in almost all cases.

CONCLUSIONS

The following conclusions have been drawn from the study.

1. Granular activated carbon columns appeared to be promising for the development of a biological film. The media offers a high surface area for biomass buildup.

2. The system demonstrates excellent potential for development into a commercially valuable sewage treatment system for specific applications. The system appears to have a short start-up time and is not sensitive to fluctuations in influent quality. The reactor can be shut down for extended periods of time. The restart period is in the range of one and a half to two hours.

3. The addition of 10-15 mg/l of sodium nitrate to the influent appears to affect the treatment process in three areas:

   a) Reduces or eliminates hydrogen sulphide production under localized anaerobic conditions and during initial start-up.

   b) Sodium nitrate alters the physical characteristics of the biomass so it will be more granular in nature and surface blinding is reduced.

   c) Bed development geometry is altered so the headloss profile across the carbon bed is considerably more linear.

4. The biological activated carbon reactor (BACR) is sensitive to loading rates and hydraulic retention time in the media. The optimum removal rates, within the range tested, appeared to be at 500 ml per minute and approximately 40 minutes retention time. Higher flow rates improved bed geometry but the BOD removal was reduced.

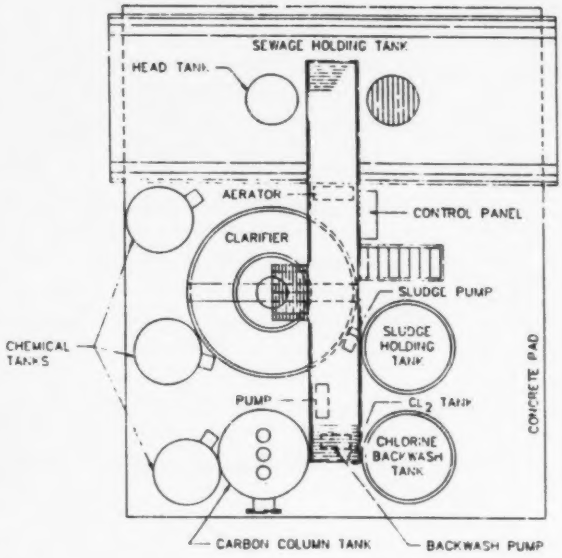
5. The presence of dissolved oxygen in the biological activated carbon reactor effluent has a strong positive effect on system performance. The level of dissolved oxygen appears to be of secondary importance. Increasing effluent dissolved oxygen values, above a trace amount, does not result in corresponding increases in BOD removal efficiency.

Based on the results of the 1988 pilot plant study a larger scale demonstration plant was constructed. This plant is now in operation at the Devon Wastewater Treatment Plant. Results to date demonstrate the inherent advantages of the biological activated carbon reactor in process reliability, simplicity of operation and compact size. The overriding advantage of the BACR is its ability to uniformly treat fluctuating and intermittent waste loads which are difficult to treat with other methods. The system is relatively insensitive to toxic influence that would render a biological plant ineffective.

Following the completion of the demonstration program, a commercial size plant will be constructed. The conceptual layout of such a system is shown in Figure 8.

The work to date has completed evaluated the performance of the BACR system by treating domestic waste under actual use conditions. It is felt that system effluent reaches or exceeds acceptable standards with BOD and suspended solids removal in the range of 95 percent. Preliminary estimates indicate that the operating costs of the BACR is in the range of 8 to 12 cents per thousand gallons.

FIGURE 6


SEWAGE HOLDING TANK
HEAD TANK
AERATOR
CONTROL PANEL
CLARIFIER
SLUDGE PUMP
SLUDGE HOLDING TANK
CHEMICAL TANKS
CONCRETE PAD
PUMP
CL_2 TANK
CHLORINE BACKWASH TANK
CARBON COLUMN TANK
BACKWASH PUMP

BIOLOGICAL ACTIVATED CARBON
SEWAGE TREATMENT SYSTEM
CONCEPTUAL LAYOUT

Figure 6

DAVID
BROMLEY
ENGINEERING
(1983) LTD.
EDMONTON · CALGARY · FT. McMURRAY

FILTER UNDERDRAINS: PROBLEMS AND NEW DEVELOPMENTS

J. Barry Hambley, P.Eng.
President
Anthratech Western Limited
Calgary AB

Many types of filters are used in Municipal Water Treatment:

- Open Top Gravity
- Low Pressure Self-Backwash Storage
- Vertical Pressure
- Horizontal Pressure
- Upflow
- Low Head Travelling Bridge

to name some of the most common types.

All filters have one thing in common, that is they have an underdrain system, the function of which is basically:

- to collect filtered water evenly (easy to do)
- to prevent passage of fine media (not, apparently, so easy to do)
- to distribute backwash flow evenly (very difficult to do)

Underdrains are called on more and more for some additional functions, for example, it is becoming increasingly desirable to have an inherent air scour capacity. Another frequent requirement is the capability to use high rate media configurations (filter coal/sand/garnet or ilmenite) and not to disrupt this layering. Another strong desire in the industry is an underdrain which does not require gravel layering, but does not eliminate gravel by opening the door to other problems like strainer failures or strainer plugging. An underdrain without gravel and without strainers would really be ideal.

There are dozens of different types of underdrains used in the industry, many proprietary, many not. These can be categorized into four main types:

1) Hub lateral
2) True header lateral (with variations)
3) Tile bottom (with variations)
4) False bottom or plenum type (with variations)

Reference AWWA Journal September 1987. "Design Options for Water Filtration" by R.G.D. Monk of Camp Dresser & McKee Inc. An excellent paper on filters and state-of-the-art underdrains.

CATEGORY 1

Hub Lateral with Variations

This type of underdrain is commonly used in small pressure filters and iron removal greensand filters up to perhaps 6-8' in diameter. It is the cheapest underdrain of all and generally a most unsatisfactory performer. By its very configuration i.e. laterals radiating out like spokes on a wheel from a central hub, one can see that the laterals are further apart the further from the centre and, on backwash, uneven backwash distribution is certain.

There is no header maldistribution problem as the hub takes the place of a header and laterals are symmetrical. The laterals must have a controlled degree of distribution, i.e. more water is needed the further from the centre as each lateral is responsible for back washing an increasing bed area. This is sometimes done by varying orifice spacing or slot size changes but problems are severe.

A variation is a double hub lateral which cuts in half the area each lateral is required to cover, but this is only a partial solution.

Differential upward back wash velocities can be compensated to a degree by graveling but one is limited in gravel depth by the side wall height restrictions.

Sometimes laterals are used with fine slots or with strainers to eliminate gravel or minimize depth of barrier gravel which ignores the importance of gravel as a diffusion layer, and uneven backwash velocities, through the media, follow.

Often to minimize cost, the laterals are made of PVC which is of course inert to corrosion, but has the bad habit of breaking at connections, in cold water particularly, if the laterals are subject to "working", which they are. Breaking of PVC laterals is very common and obviously disastrous.

In short, hub lateral underdrains, though very common, are about the worst underdrain one could use and responsible for much of the many problems users have with Greensand filters and small pressure filters generally.

CATEGORY 2

True Header Lateral (with Variations)

These types are reasonably common in vertical and horizontal pressure filters and in some open top gravity filters and frequently used in "packaged" water plants.

In the simplest form these consist of a pipe header down the centre line of the filter with pipe laterals down each side of the header. Usually the pipe laterals have orifice holes, with graded gravel layering to block off fine media passing through, and to achieve a degree of backwash flow spread.

Variations on the theme are many. The configuration sometimes is a header down the side with laterals off one side of the header. The header may be external to the filter and may be a concrete conduit with laterals running through the wall and across the filter. Sometimes the laterals are slotted, or wrapped with stainless steel mesh, or have strainers, or are of wedge wire or plastic wedge or plastic cloth wrap. Some designs have air scour capability usually using strainers with air metering tube extensions.

A common older variation is the Wagner Underdrain comprising a down-the-centre box conduit, usually of transit, with transit pipe laterals. Between the laterals pre-cast blocks were installed, shaped to deflect backwash flow from the laterals upwardly through slots in the concrete blocks. The whole thing graveled, and the flat top of the centre conduit having strainers or nozzles usually, to eliminate this area as dead space.

As mentioned, some of these designs have air scour capability. Many do not and if air scour were required this would usually consist of a separate air header with laterals, usually at the near bottom of the silica sand media to avoid blowing air through the graveling.

These types of underdrains have been used for many, many years and while some of them operate reasonably trouble free, many are very troublesome being particularly prone to uneven backwash distribution in both header and laterals, air problems, gravel disruption, etc.

CATEGORY 3

Tile Bottoms (with Variations)

The most common types are Leopold fired clay and lately Roberts and others with nearly identical products. Gravel layering is required. Generally use surface and/or subsurface washers as cannot air scour with these blocks.

Variations are concrete block, Leopold "Universal" plastic block, which has air scour capability, and, an old design, the Miller Block.

Basically all of these blocks are placed in rows, essentially forming laterals, with the lateral rows open to a sunken concrete flume across the front end or down the centre line, serving as the filtered water collector and backwash water "header".

Interestingly enough, the Leopold block is a very old design, and for a period of time in the 1950 - 1965 era lost out heavily to false bottom designs with strainers, which eliminated graveling. Problems with these resulted in a swing back to the Leopold block. In the past few years the Leopold "Universal" plastic block with air scour capability has been promoted instead of the clay block.

These systems are pretty well limited to open top gravity filters and not applicable in pressure or low pressure self backwash storage types. Many filters of the tile bottom type run trouble free year after year. Others are not so fortunate, air upsets of the gravel being one of the most common troubles.

Block bottom designs have no header correction for even distribution into the block row "laterals", and header to lateral maldistribution can be very significant. The block row laterals in the Leopold design have a double distributing conduit design so maldistribution in the "laterals" themselves is not much of a problem until the run length becomes quite long.

Installation is tedious and costly.

CATEGORY 4

False Bottom [or Plenum] Type with Variations

There are a great variety of designs in this category. Some require graveling, some do not. Some have strainers, some do not. Some can air scour and some can't. We will run out of space if all of these are described in any detail, but will list some of them:

- Wheeler bottom - concrete block or cast in place. Uses ceramic balls in inverted pyramid depressions.
- Cast in place - Patterson-Candy, Eimco, Ecodyne, and others.
- Precast block on pedestals - Infilco Fre-flow, Eimco, Ecodyne and others.
- Asbestos cement or composite block on legs - Infilco Fre-flow and others.
- "Tee-pee" design - precast concrete inverted v shape.
- Porous plate design - Carborundum and others.
- Double dish - used sometimes in vertical pressure filters.
- Painted steel, or galvanized, false bottom plates with strainers - various suppliers. [All self backwash storage low pressure filters have this type of underdrain.]

Further in this paper we will be discussing indirectly the various foibles of these designs.

One would think that with work on underdrains having gone on for well over 150 years that all problems in this area would be solved. Not the case. There is no doubt that some of these underdrains are much better than others but problems with underdrains are very frequent. Some of these many problems are:

- Gravel upsets resulting in fine media passing.
- Structural failures - corrosion/lifting of blocks/inversion of Wheeler balls
- Strainer failures
- Strainer plugging
- Uneven backwash distribution - mud balling and channelling and short circuiting.
- Poor air scouring distribution
- Explosive air release - inability to cope with backwash air entrainment or air released due to negative head operation.
- Media upsets - some types of filters cannot use high rate media.

- Strainers - jetting and rolling action - often requiring graveling to provide distribution of backwash flow.

Strainers were developed to eliminate graveling so that is a full circle indeed, when graveling is required to minimize jetting.

Basically then we set out to develop and have developed a filter underdrain, the "Newtech" underdrain which overcomes the multitude of problems which current underdrains have. Patents applied for in the U.S. and Canada. Figures 1, 2 and 3 illustrate the following description.

General Notes Re; Figures 1, 2 and 3

This demonstrator is made entirely of 304 Stainless Steel. The use of other materials for example 316 Stainless Steel for Activated Carbon columns, is quite evidently possible.

Hold-down bolts will typically be Stainless Steel Hilti or equal adhesive set bolts, though expansion anchors may be used.

The underdrain is entirely independent of the walls of the filter.

The design is flexible to suit any configuration of filter—square, rectangular, round; gravity or pressure. A variation of the computer program is used to size flume header cover plate orifices when discharge is to flute laterals of varying length as in a circular filter.

Air addition to the flute air chambers can be from above or from an air header below in the flume with air proportioned upwardly through the air division plate. Both designs are available to suit.

On small filters and ion exchange units, the flute laterals are modified to exclude the division plate shoulder with the profile wire sections spanning flute laterals tops.

Not included in the demonstrator but an essential part of the underdrain air vent holes typically 3/16" diameter on 4' centres through the flute side and flange of the air division plate to provide for controlled release of any air tapped or introduced with backwash water. If air pockets occur in headers or laterals these have a very marked effect on the hydraulic distribution pattern. This is a far more important function than it might appear as trapped air unpredictably alters backwash water distribution.

This is an effect often overlooked in underdrain design but most critical to the operation.

FIGURE 1

Showing Newtech Underdrain on a simulated concrete base. A channel in the concrete base, visible on the left, serves as the filtered-water-out backwash in "header". The channel is covered with a plate, in which orifices are cut so backflow flow is upward into crossing flute laterals. The orifices vary in size, predicted by a computer program, to evenly meter the backwash flow into the flute laterals by compensating for variations in orifice discharge coefficient due to crossflow velocities. One such orifice is visible on the plate trough towards the right side of the photo.

One the left side is shown a standard filter side edge treatment. The flute end treatment is a stainless steel plate with a neoprene seal held on by bolted clip angles as shown. Alternately the end plate is welded to the flute lateral to eliminate the neoprene seal.

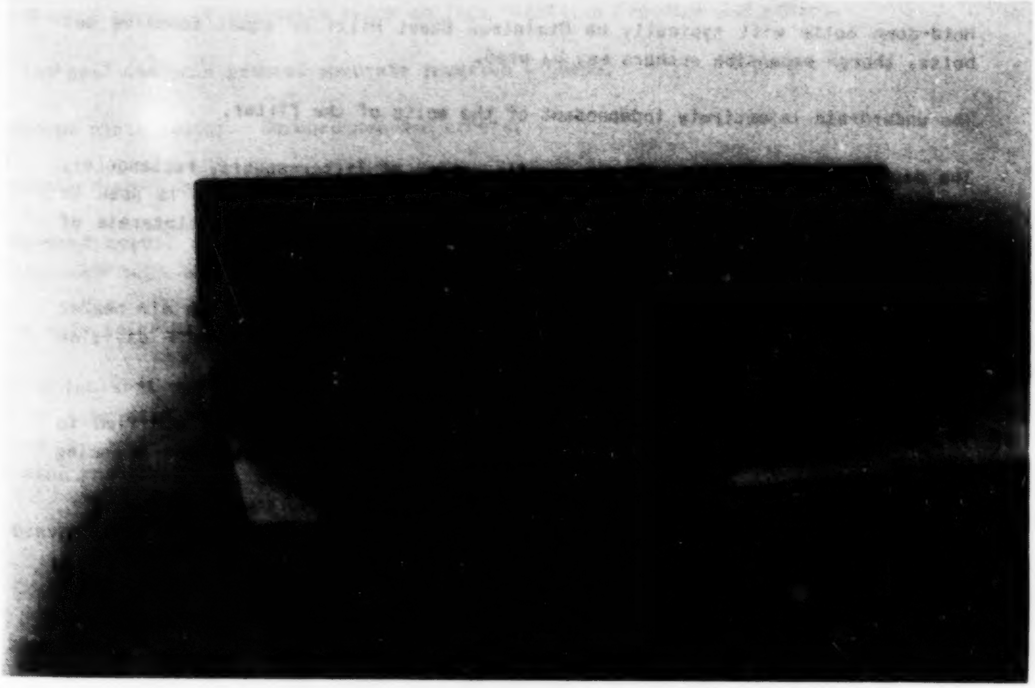


FIGURE 2

Shows the second flute lateral in place. Clearly shown are the flute flange hold-down slotted channel pieces.

Hold-down is done this way to provide 2 degrees of freedom in positioning hold-down bolts to miss rebar in the concrete base. Rebar is located using a pakometer (a sensitive metal stud finder).

The flute lateral backwash water outlet orifices are clearly shown. These vary in size along the flute lateral again to compensate for discharge coefficient changes due to crossing velocities.

Note the partitioning of the flute lateral by a plate dividing the lateral into an upper air channel and lower water channel. By such division the problem of wave action which accompanies air scour air addition into a lateral not so divided is eliminated.

Air outlet orifices are shown along the flute lateral sides designed to pass the required volume of air at a 3-4" pressure drop.

The air scour division plates are bolted in place in this demonstrator for purposes of ease in disassembly. Typically these plates would be stitch welded rather than bolted.

Note that this design is flexible for air scour followed by water backwash or for the increasingly popular mode of operation of air scour simultaneous with water backwash.

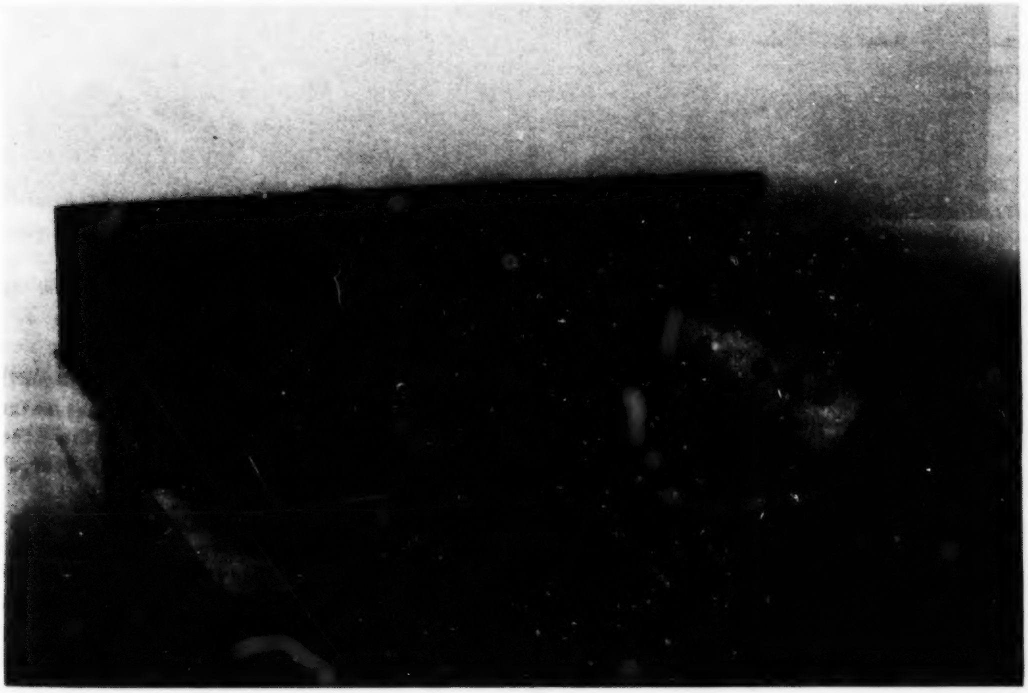
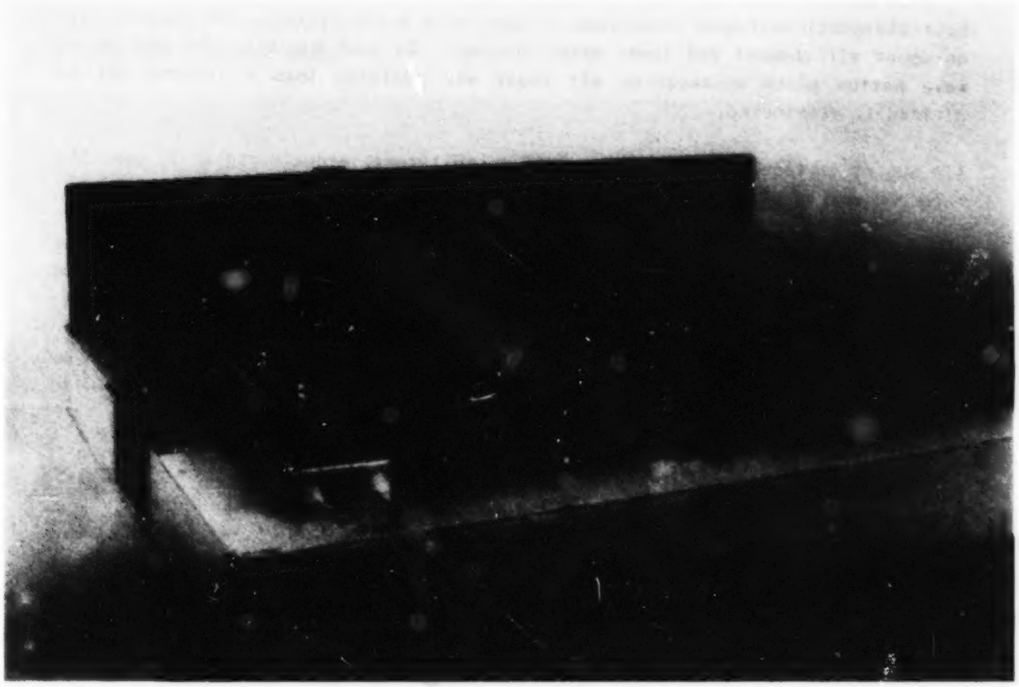


FIGURE 3

Shows a profile wire trough cover plate with slot gap to suit (this demonstrator has .4 mm slot sizes) to prevent ingress of the filter media. The edge treatment shows a half cover plate and shows how these are bolted in place. (These bolts may also be installed with pop rivets, or self tapping screws or commonly will be field stitch-welded to the flute laterals).

The design shown requires no gravel layering, which will be by far the most common mode.



Materials of Construction

All stainless steel typically 304 as standard. Can readily be made of 316 SS where conditions warrant (carbon contactors for example). Many other materials could be used e.g. galvanized or painted steel, aluminum, high density polyethylene, fibre glass, etc. but stainless steel is standard to eliminate any corrosion concerns in what is a very sensitive area of a filter.

The physical design becomes one where stainless steel, which is not an easy material to work with, can be used cost effectively.

Structural Strength

Flexible for the service by using different gauge thickness of stainless and varying the supports and spacing for the profile wire grids. Typical design for open-top gravity filters is 10 psi. Where conditions warrant and particularly in pressure filters, standard designs are readily available for 15/30/80/150 psi and higher if need be.

Installation

On flat concrete floor. No plenum or false bottom is required. The underdrain is basically a header/lateral variation. The header typically is a flume or conduit formed in the concrete either crossing the inside front of the filter cell or running the centre length or could be an external conduit or pipe header feed. In circular filters the flume spans across the diameter. A stainless steel flume cover plate is used with orifices to distribute flow proportionally to each flute lateral. Crossing the flume are the flute laterals.

Distribution Hydraulics

The key to even distribution of backflow flow is to make two corrections. First the flume cover plate orifices are sized using a computer program to compensate for variance in orifice discharge coefficients due to cross flow velocity.

This is a water momentum effect. The discharge through an orifice when the crossing velocity is say 4 fps is markedly less than when the crossflow velocity is say 2 fps across an orifice of the same size. This is why maldistribution occurs in a filter header or flume, with the back end of the filter receiving higher flows than the front end. This can have a very drastic effect on filter performance as one can imagine in a filter cell say 30' long with the front third backwashing at 10 USGPM/square foot and the back third backwashing at 20 USGPM/square foot.

The effect of a Category 4 false bottom or plenum, which takes the place of a header or flume, is to greatly reduce the cross flow velocity so to eliminate varying coefficients of discharge up through the false bottom orifices or strainers or whatever. Often however the false bottom is not made deep enough to eliminate velocity effects.

In passing, we remark also that violent inlet turbulence can cause a maldistribution i.e. a greater than anticipated flow in the turbulent areas. The combination of a narrow false bottom with many "supports" or "legs" to contribute to local turbulence, can be very troublesome.

Another unrecognized contributor to backwash maldistribution in a number of filter underdrain types is trapped pockets of air introduced in the backwash or from negative head operation. This can lead to orifice cross flow velocities which are variable and is often the reason why identical filter cells exhibit different backwash characteristics.

Often with the Newtech underdrain we install a flow straightener in the backwash line entrance to reduce swirl turbulence.

The above mentioned variation in orifice discharge coefficients with cross flow velocity is not easy to determine. We have determined the variance using hydraulic models, tested under different conditions of temperature, velocities, orifice pressure drops, recovery of energy, etc. and developed a computer program to determine the variance in flume cover plate orifice sizing to ensure even output flows.

In circular filters, where the flute lateral lengths vary, a modification of the program is used to size plate orifices for proper distribution - essentially a controlled maldistribution as it were.

We are not aware of any other underdrain which corrects for header or flume maldistribution and this we have found in hydraulic test work is a major source of maldistribution.

In flowing along the flute laterals themselves a similar situation of cross flow effect on orifice discharge occurs. A different set of orifice discharge coefficients is applicable as no orifice discharge energy is recovered. These are run in another computer program to set the variance in discharge orifice sizing along the lengths of the flute laterals.

Thus backwash flow is totally even throughout the filter by correcting the "header" as well as correcting the "laterals".

The Newtech underdrain is unique in providing very even backflow flow distribution on a custom design basis.

Again, in starting out with flat stainless sheets, the orifices are punched rather than drilled (drilling stainless is difficult) and a computerized punch press is used to easily make the varying orifice sizes on centres to suit. Incidentally, orifice sizes are set for maximum backwash rates of flow. If backwash water flow rates are reduced seasonally, as is common, distribution is not affected, if the drain is done on maximum rates.

Hold-down

The Newtech underdrain is fastened down to the concrete base using adhesive set stainless steel bolts. A variety of cinch, expansion or self drilling anchors could be used but adhesive sets are preferable and standard.

The flute lateral flanges are set with a spacing gap of 2 to 2-1/2". On retrofits we do not want to drill into rebar. A "pakometer", a very sensitive stud finder is used to locate embedded rebar. The adhesive set bolts are then drilled to miss the rebar. The flanges are held down with a slot top channel piece, this method giving plenty of flexibility on hold-down bolt location so to avoid rebar.

End Seals

Sealing of the ends of the flute laterals is done using a stainless steel back plate with a neoprene strip, the whole thing bolted to the flute lateral with stainless steel clips and bolts.

Alternately the end plates are welded on.

Air Scour

North American filter practice has been, where warranted, to air scour a filter at rates of 3-4 SCFM/square foot for 3-5 minutes, prior to hydraulic backwash. This has typically been done on dual media filters where depth filtration occurs and surface washers cannot get at the deeply embedded solids. Sub surface washers just above the filter coal/sand zone of mixing also have a fairly limited zone of effect. Hydraulic backwash alone is quite a gentle action and the much more violent action of air in loosening adherent sticky sludge like algae, clay, or lime softening sludge is often very useful in keeping the entire bed clean.

The Newtech underdrain is readily adapted to air scour in a rather unique way. The flute lateral is fabricated with a division plate to divide the lateral into an upper air area and lower water area. This is a very important innovation. In two phase flow, at even very low air velocities a wave action occurs and at a little higher velocity a slugging action develops. Both wave action and slugging lead to a very erratic distribution of air. This occurs in false bottom filters commonly, and in other types, like the Leopold Universal for example, utilizing the same conduit for air in contact with water whether the water is flowing or not. Hence in the Newtech design a positive division of air and water to positively eliminate the effect of wave action, and to ensure even air distribution.

Air is introduced into the upper air "conduit" from either above or below to suit. Both designs are available. The air outlet holes are just above the division plate and are designed to meter air at approximately 3" back pressure. Air introduction quickly displaces all the water out of the air conduit section, the air then exits freely. With profile wire grids the air rises from each air outlet row in an approximate 2" wide, then spreading, curtain wall which provides a very effective air scouring action through the media.

Another mode of air scour which is becoming increasingly popular (Los Angeles Aqueduct Plant, Salt Lake City Water Plant Expansion) is European in origin. In this mode air scouring is simultaneous with backwash water addition. Media bed expansion is only 5-6%, caused by the air, with just enough backflow flow, typically 7-8 USGPM/square foot, to provide enough sweep velocity to remove the filtered-out solids.

Backwash requirements are half that of "conventional" filters. Special consideration is given to media type and sizing to avoid floating off filter coal. Often mono-media i.e. sand only or filter coal only (as at the Aqueduct Plant) is used to avoid a final high backwash rate restratification step required for dual media.

The Newtech underdrain is perfectly adapted to air scour simultaneous with hydraulic backwash.

Air Venting

The flute laterals have small air vent holes, typically 3/16" diameter on approximately 3-4' centres through the flute side immediately below the air division plate to provide a means of controlled vent release of trapped air, from air release when negative head conditions exist, or introduced from the backwash system.

Most underdrain systems have no way of coping with air entrainment or explosive air release. Trapped air can seriously effect backwash distribution and explosive air release is violently destructive to gravel layering, and float off losses of filter coal.

Trough Grid

The standard shape of flute lateral can be used as is with gravel layering, using the initial gravel layer of such a size that coefficient of discharge from the orifices, which now see adjacent gravel, is not affected.

Where air scour is used with gravel layering, an inverse gradation or "hour glass" design is used to maintain the gravel layer position. Air scour is very disruptive to gravel layering. The air bumps the gravel causing a slight job and finer gravel works it way down to wedge the larger gravel upwardly. In other words the large gravel works its way up, the finer gravel down and an improperly designed gravel bed quickly becomes disrupted.

As mentioned this effect can be compensated for by using inverse gradation sometimes with garnet filling the void of the top two coarse layers, sometimes using "heavy green" gravel of high density for some or all of the layers to minimize shifting.

However, one of the main features of the Newtech underdrain is the use of stainless steel profile wire grids to permit a design that has no gravel. The profile wire is gaped to suit the media size used eg. .5 mm, .4 mm, .25 mm.

Grid position is above the air and water orifices and above the scour outlet orifices. The profile wire and supports are stainless steel, the supports are varied from round rod for low pressure filters to 1/6 - 3/16 x 1" flats on centres to suit for higher pressure.

The standard design has 55% grid surface coverage which is perfectly fine for most filters, and a far higher surface area coverage than other designs. For other applications where even higher surface area coverage is desired, like small pressure filters or ion exchange units the design is readily modified so that 90+% of the surface is covered with profile wire.

In summary then, the Newtech Underdrain has many advanced features:

- Stainless steel construction
- Very strong
- Air scour capability (flexible air scour followed by backwash or air scour simultaneous with backwash with excellent air distribution)
- Excellent backwash distribution characteristics
- Flexible - can run gravel design or no gravel requirement
- Simple and fast to install - requires no grouting
- Controlled venting of trapped air
- Can use high rate media configuration / or dual media / or mono media
- Shipping costs low
- No false bottom required (cuts down on filter wall depth)
- Very flexible for new filter sizes and configurations
- Easy retrofits of troublesome filters - open top gravity, vertical and horizontal pressure filters, etc.
- No strainers
- High % of surface coverage
- Installation is simple and is independent of the filter walls
- Low profile - typically height of flute laterals 5 - 6"
- Low Pressure Drop - the ΔP through the underdrain is designed to suit by changing diameter of the plate orifices and of the flute lateral orifices. An overall ΔP of 6-8" is typical.
- Very cost effective

USE OF DEMONSTRATION SCALE SOLIDS CONTACT SLURRY RECIRCULATING CLARIFIERS (SCSRC) IN THE EVALUATION OF POWDERED ACTIVATED CARBON (PAC)

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INTRODUCTION AND BACKGROUND

Activated carbon adsorption has been an important process for taste and odour control in the treatment of drinking water for many decades. Application can be in either powdered or granular form. Granular activated carbon (GAC) is utilized in a column format with the water flowing through it. Alternatively, powdered activated carbon (PAC) is dispersed through the water from which it must subsequently be removed via sedimentation or filtration.

In existing drinking water treatment facilities, the addition of PAC to the treatment scheme involves minimal additional capital costs and thus provides an obvious advantage over GAC implementation. However PAC is hindered in some cases by a short contact time and by a dose restricted to 25 mg/L if it is to be handled by a downstream filtration step. This would tend to limit the application potential of PAC.

Since PAC can be utilized in variable dosage in the context of existing facilities, it is attractive for locations with periodic or seasonal problems. Such is the circumstance of water quality concerns in the City of Edmonton. This City is served by two treatment installations, E.L. Smith and Rossdale.

The E.L. Smith installation utilizes solids contact slurry recirculating clarifiers (SCSRC). Fluidized bed PAC adsorbers, with addition of PAC to the floc blanket have the advantage of long residence time in the blanket and good contact between floc and water flowing through, enabling equilibrium to be approached. Prior studies have indicated tangible benefits of this application.[1,2,3,4,5,6]

To evaluate PAC as retained in a SCSRC floc blanket, the accumulation profile and adsorptive behaviour of the carbon have to be characterized. Studies to date have not quantified these parameters in solids contact clarifiers.[1,2,3,4,5,6] If research were to confirm enhanced retention and contact of PAC in an SCSRC, this could constitute a cost effective alternative to GAC under certain treatment requirements.

Pilot Plant Operation

To evaluate variation in carbon application on plants performance, it was necessary to have a pilot facility, since upset conditions not amenable to regular plant operation could potentially arise during these studies. The pilot plant used was a trailer mounted unit developed at the Alberta Environmental Centre, Vegreville. Figure 1 shows the plant layout and Figure 2 shows the process schematics. The main components of the pilot plant used for this investigation were the two clarifiers. These were operated in parallel, with one serving as a control (sludge blanket without PAC) and the other as the variable unit (sludge blanket with range of PAC doses). A detailed cross-section of the solids contact slurry recirculating pilot clarifier is shown in Figure 3.

FIGURE 1

Pilot Plant Layout

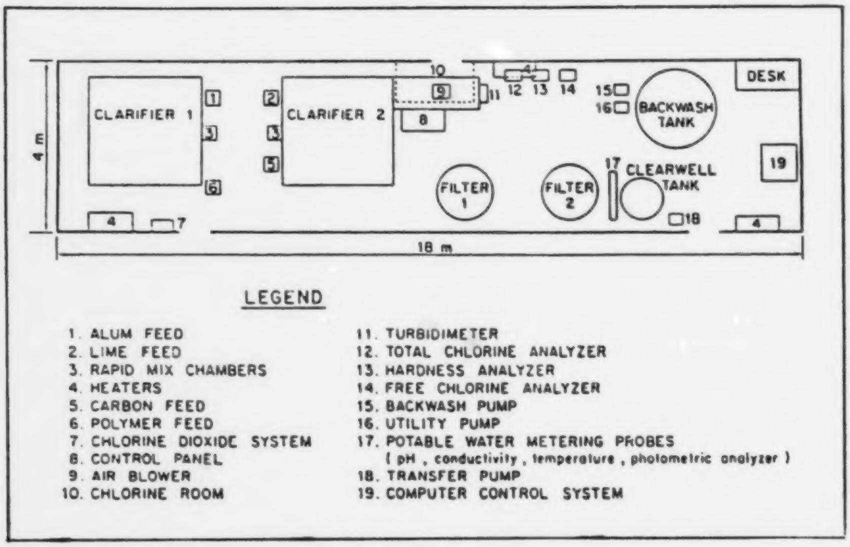

CLARIFIER 1
CLARIFIER 2
1
2
3
3
5
6
4
7
8
9
10
11
12
13
14
15
16
17
18
19
4
DESK
BACKWASH TANK
CLEARWELL TANK
FILTER 1
FILTER 2
4 m
18 m
LEGEND
1. ALUM FEED
2. LIME FEED
3. RAPID MIX CHAMBERS
4. HEATERS
5. CARBON FEED
6. POLYMER FEED
7. CHLORINE DIOXIDE SYSTEM
8. CONTROL PANEL
9. AIR BLOWER
10. CHLORINE ROOM
11. TURBIDIMETER
12. TOTAL CHLORINE ANALYZER
13. HARDNESS ANALYZER
14. FREE CHLORINE ANALYZER
15. BACKWASH PUMP
16. UTILITY PUMP
17. POTABLE WATER METERING PROBES
(pH , conductivity , temperature , photometric analyzer)
18. TRANSFER PUMP
19. COMPUTER CONTROL SYSTEM

FIGURE 2

Pilot Plant - Process Schematic

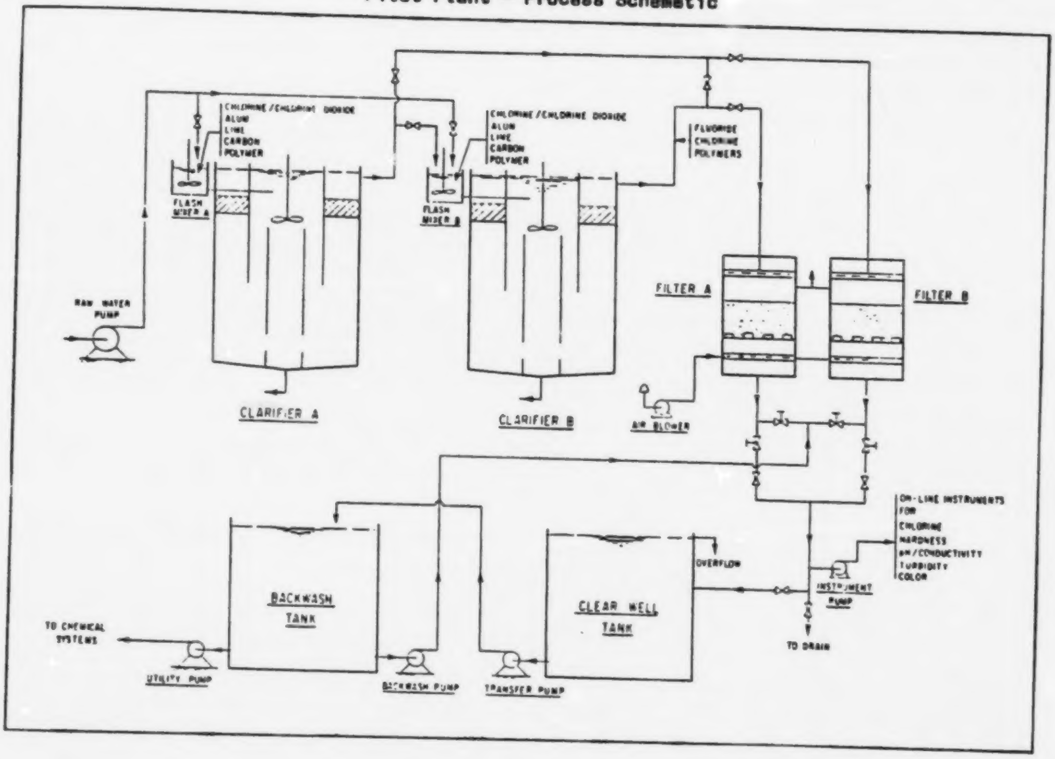
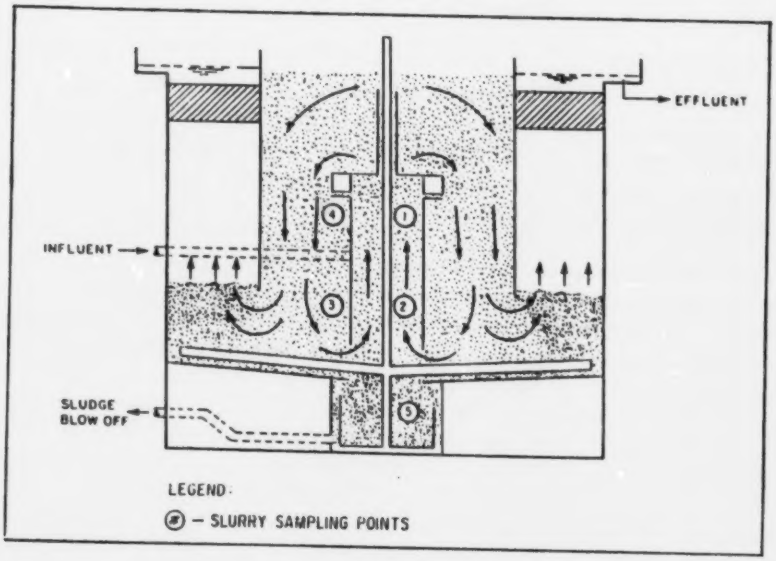

CHLORINE/CHLORINE DIOXIDE
ALUM
LIME
CARBON
POLYMER
FLASH MIXER A
CHLORINE/CHLORINE DIOXIDE
ALUM
LIME
CARBON
POLYMER
FLASH MIXER B
FLUORIDE
CHLORINE
POLYMERS
RAW WATER PUMP
FILTER A
FILTER B
CLARIFIER A
CLARIFIER B
AIR BLOWER
ON-LINE INSTRUMENTS FOR
CHLORINE
HARDNESS
pH/CONDUCTIVITY
TURBIDITY
COLOR
OVERFLOW
INSTRUMENT PUMP
BACKWASH TANK
CLEAR WELL TANK
TO CHEMICAL SYSTEMS
TO DRAIN
UTILITY PUMP
BACKWASH PUMP
TRANSFER PUMP

FIGURE 3

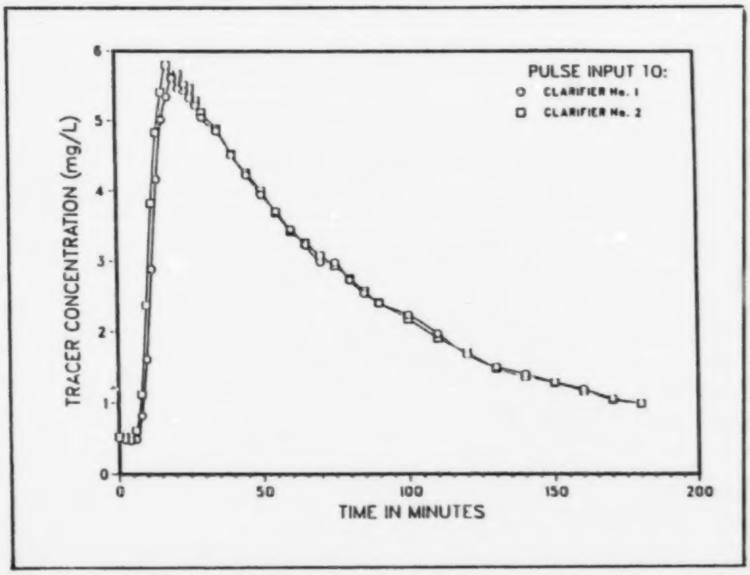
Pilot Clarifier Cross Section


EFFLUENT
INFLUENT
SLUDGE BLOW OFF
①
②
③
④
⑤
LEGEND:
Ⓧ – SLURRY SAMPLING POINTS

To ensure that the two pilot clarifiers had comparable hydraulic characteristics, tracer studies were carried out. This was necessary since the basis of the experimental runs was to compare performance of carbon added to one clarifier, while the other was used as a control. Figure 4 shows the relative performance of the two units. A retention time varying between 63 and 67 min for both clarifiers at 150 L/min flow was obtained.

FIGURE 4

Response Curve In Clarifiers To Tracer Pulse Input



The other comparison was to ensure that the pilot scale clarifiers' operating performance was similar to the full scale E.L. Smith clarifier performance. Using Aluminex® as a coagulant for the pilot plant, its optimum operational performance came within 10% of the full scale plant as summarized in Table 1. Using alum as the coagulant, matching performance was difficult to attain. This is illustrated in Figure 5.

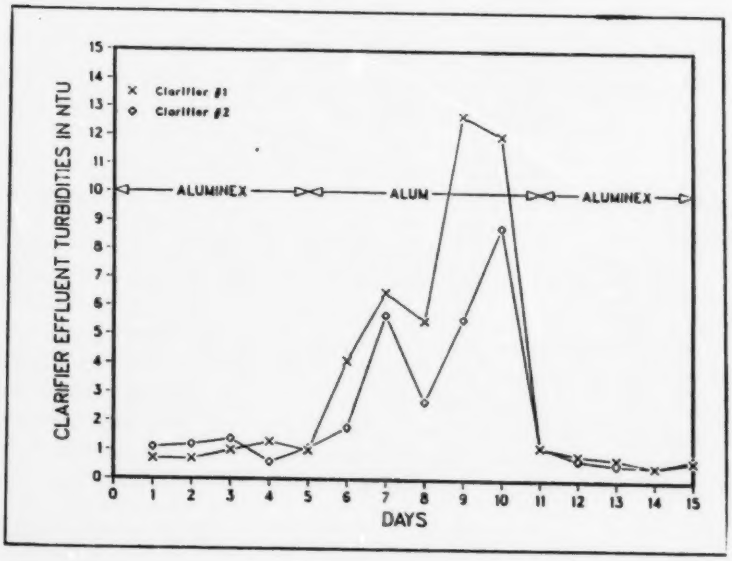
In the interests of developing a good concentration of solids slurry to establish effective clarifier performance in an expedited manner, it was necessary to develop an accelerated start-up procedure. Initial procedures utilized bentonite clay for seeding the pilot scale clarifiers which provided adequate suspended matter to enable quick start-up. However, over a period of 4 to 5 days of continuous operation the slurry would dilute and disappear through the clarifier effluent. Alternative seeding procedures were investigated and a procedure using dry [powdered] calcium carbonate was selected. This lead to the formation of a high quality slurry which was readily maintained during the operation of a pilot plant run.

Routine plant operation was monitored and control strategy based on turbidity [influent and effluent] and V/V solids percent in the slurry.

TABLE 1
Comparison of Operating Data
Pilot Plant vs. E.L. Smith Plant

DESCRIPTION	PILOT PLANT	E.L. SMITH PLANT
PEAK DESIGN DATA		
Flow	205 L/min	200 MLD
Clarifier Rise Rates	5.3 m/hr	5.6 m/hr
Residence Times		
Clarifier	47 min	88.4 min
Solids Contact Chamber	14 min	18.2 min
Rapid Mixer	15 sec	0.04 sec
Volumes		
Total	9.7 m3	12,488 m3
Draft Tube	0.08 m3	118 m3
Inner Mix Zone	2.85 m3	2525 m3
Rapid Mix	0.05 m3	5 m3
COMPARATIVE PERFORMANCE DATA		
(@ Effluent Turbidity < 2.0 N.T.U.; Water Temperature = 0.5°C)		
Flow	150 L/min	150 MLD
% of Peak Design Flow	73%	75%
Retention Times	64 min	115 min
Rise Rates	3.9 m/hr	4.2 m/hr

FIGURE 5
Coagulants – Effect On Clarifier Performance



Laboratory Procedures

One of the most challenging aspects of this study was the development of a method to analyze the concentration of activated carbon in the aluminum hydroxide based clarifier. An extensive literature review did not disclose an existing method to perform the analysis.

BASIS FOR METHOD DEVELOPMENT

The method was developed based on consideration of the contents of the clarifier slurry: (i) aluminum hydroxide or polyaluminum chloride (coagulant); (ii) polymer (organic coagulant aid); (iii) other inorganics; (iv) absorbed or retained organics; and (v) PAC.

The most direct way of quantifying the presence of carbon is by weight, and therefore a gravimetric analysis approach was considered appropriate[7]. On the basis of the main constituents outlined above, the following analytical steps were proposed:

(i) Acid Dissolution

Acid dissolution was considered the most effective method to separate the carbon from the aluminum hydroxide precipitate. The aluminum hydroxide precipitate dissolves at low pH.

(ii) Filtration

Once the aluminum hydroxide is dissolved, the remaining suspended mass including PAC, can be separated from the solution phase by filtration.

(iii) Volatilization (Drying)

Subjecting the filtered suspended materials to drying will remove any remaining water and low boiling point (volatile) organics.

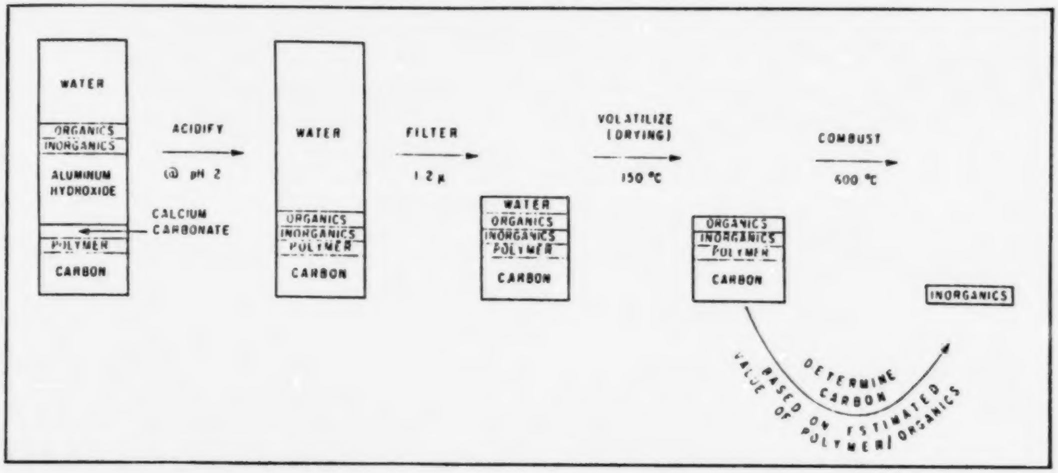
(iv) Combustion

The final combustion procedure will convert the organic fraction of the PAC into CO_2 and enable quantification of the amount of PAC from the pre-determined ash content of that particular carbon.

Figure 6 provides a pictorial representation of the effects the above procedures would have on the solids slurry sample.

On the basis of the above, a method for determining the carbon concentration was successfully developed. Known amounts of all of the above constituents were mixed to stimulate a clarifier slurry and subjected to the procedure. The PAC concentrations determined by the method were found to be within 10 percent of the actual amounts[8]. During experimental runs, a correction factor to account for the presence of polymer and any retained inorganic solids was determined by using the control clarifier slurry results. However, it was observed that the correction factor changed only insignificantly during the course of the experiment. Therefore the correction factor derived just prior to the start of PAC addition was used for correction through the experimental run.

FIGURE 6
Pictorial Representation of PAC
in Slurry Determination Analysis


WATER
ORGANICS
INORGANICS
ALUMINUM HYDROXIDE
POLYMER
CARBON
CALCIUM CARBONATE
ACIDIFY
@ pH 2
WATER
ORGANICS
INORGANICS
POLYMER
CARBON
FILTER
1.2 µ
WATER
ORGANICS
INORGANICS
POLYMER
CARBON
VOLATILIZE (DRYING)
150 °C
ORGANICS
INORGANICS
POLYMER
CARBON
COMBUST
600 °C
INORGANICS
DETERMINE CARBON
BASED ON ESTIMATED VALUE OF POLYMER/ORGANICS

QUALITY ASSURANCE AND CONTROL

Quality control of the PAC in sludge analysis was carried out in two ways: (i) each run of the analysis consisted of 12 samples of which 2 were blanks. The blanks (deionized water) were subjected to the same gravimetric analysis procedure as the slurry samples, i.e. acidification, filtration, volatilization, and combustion. If the blank samples gained or lost more than 10% by weight at any stage of the analysis, the entire run was discarded. During one of the experimental runs, this quality control program helped identify a contaminated acid solution used as reagent; (ii) random samples were sent out for independent analysis to the Alberta Environmental Centre for confirmation of the results.

Method for Determination of PAC in Clarifier Slurry[8]

(i) Apparatus

- Analytical balance accurate to at least 0.001 g.
- Glass fibre filters, GF/C or equivalent (90 mm diameter or as required to fit Buchner funnels).
- Laboratory oven capable of 150°C.
- Muffle furnace capable of 600°C.
- Muffle furnace tongs.
- Desiccator sufficiently large to hold four or more glass fibre filters in petri dishes or similar containers.
- Crucibles.
- Petri dishes.
- Tweezers.
- Vacuum pump (filtering flasks).
- Measuring cylinder.
- 250 mL beakers

(ii) Reagents

- Distilled or deionized water or equivalent.
- Sulphuric acid, 9% w/w.

(iii) Procedure

(a) Filter Preparation (use tweezers at all times when handling filters)

- Dry filters in petri dishes overnight at 105°C.
- Place the dried filters in petri dish, and place into the desiccator until cool (about 10 minutes).
- Weigh filters* only to four decimal points. This is the tare weight.

* filters required would be 3 replicates per sample plus two blanks.

(b) Acid Dissolution

- Using a measuring cylinder measure 3 x 100 mL of the slurry sample and pour into a 250 mL beaker (3 replicates each). Also prepare 100 mL distilled water for 2 blank replicates for each round of analysis.

- Pour 10 mL of 9% w/w sulphuric acid solution into the beaker contents. Mix the contents thoroughly and allow to stand for 3 to 5 minutes.

(c) Filtration

- Filter each sample using Buchner funnel and vacuum flask, employing a previously tared glass fibre filter. Exercise caution during the filtration to ensure that all the solids are trapped on the filter, and none stuck to the sides of the funnel. Wash down visible solids remaining in the beaker with small quantities of water.

(d) Volatilization (Drying)

- Place filters in petri dishes and dry at 150°C for one hour or more.
- Then place in desiccator until cool (10 minutes or so).
- Weigh filter and retentate. This weight is the initial retentate plus filter weight.

(e) Combustion

- Preheat muffle furnace to 600°C (Approximate - not calibrated).
- Fold the filters and place them into the crucibles.
- Place crucibles into the muffle furnace for one hour.
- Remove from muffle furnace and place on a heat resistant surface (e.g. brick) for 5 minutes.
- Place in desiccator until cool (15 minutes).
- Weigh the filter plus ash. This gives the final weight.

(f) Calculation

Ci = undried PAC (initial) weight, g.
Cf = PAC weight acid washed, filtered and dried, g.
Cc = PAC weight loss due to combustion.
Si = sludge (initial) weight, g.
Sf = sludge weight acid washed, filtered and dried, g.
Sc = sludge weight loss due to combustion.
Ts = total solids weight (initial), g.
Ds = total dried solids weight, g.
WL = total weight loss due to combustion, g.
Fc = filter weight loss due to combustion, g.
Kc = fraction of combusted PAC
Ks = fraction of combusted sludge

Assume:

Cf = Ci (no PAC loss during acid wash and drying)
Ds = Cf + Sf = Ci + Sf .... (1)
WL = Cc + Sc + Fc .... (2)
Kc = Cc/Cf = Cc/Ci .... (3)
Ks = Sc/Sf .... (4)

Solving for Ci:

$$Ci = \frac{(WL - Fc) - KsDs}{Kc - Ks}$$

Typical values observed during this research investigation were:

Kc = 0.898, Ks = 0.143; Fc = 0.0029 g.

OBSERVATIONS

The utilization of the described method for the quantification of carbon content in clarifier sludge proved effective in evaluating clarifier/PAC accumulation performance.

Experiments undertaken at 10 mg/L and 25 mg/L carbon feed rates demonstrated a relationship between carbon feed and maximum accumulated carbon concentrations in the clarifier sludge. For the maximum feed rate (25 mg/L), a maximum accumulated concentration of almost 4,000 mg/L was observed (a ratio of 160 for accumulation/feed). Approximately the same ratio resulted from the 10 mg/L trials.

Consequently, it was demonstrated that during periods of low turbidity, consistently high values of carbon accumulation in sludge could be achieved. However, during the onset of high turbidity episodes, this situation was dramatically altered with rapid declines in the carbon/sludge ratios. This can be attributed to the greatly increased need for clarifier underdrain blowoff to maintain satisfactory V/V levels. During these episodes, carbon replacement rates can not maintain those quantities of carbon lost due to increased clarifier draining. Consequently, a new and vastly reduced carbon to sludge ratio is reached based on the new carbon input/loss conditions.

It was also observed that the concentration of PAC in the pilot solids contact slurry recirculating clarifiers was evenly distributed through the entire clarifier up to the tube settlers, suggesting this to be an efficient means of activated carbon contact. Detailed results of the adsorption performance of PAC in these sludge blankets are presented elsewhere.[8,9]

CONCLUSIONS

The current trials have demonstrated the capability of achieving much larger carbon concentrations than feasible in a single pass system. However, the raw water conditions must be such as to minimize the need for frequent clarifier draining. This situation would be consistent with various water supplies depending on seasonal water quality variations. The even distribution of carbon through the clarifier as observed would ensure an effective and extended contact period. Therefore, this type of procedure should be considered as a viable option where plant facilities and raw water quality allow.

ACKNOWLEDGEMENT

This study was a joint participation of the City of Edmonton's Environmental Services Department, the Environmental Technology Division of the Alberta Environmental Centre, and the University of Alberta Department of Civil Engineering. The discussions and helpful suggestions from Dr. Vern Snoeyink of the University of Illinois are appreciated.

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INTEGRATED CONTROLS FOR WATER AND SEWAGE TREATMENT FACILITIES

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INTRODUCTION

The Town of Cardston, Alberta has recently upgraded its municipal water supply with the completion of a fully automated water treatment plant (WTP). The Town, located in southwestern Alberta is typified by large residential lots with much lawn and garden area. Residential water services are not metered. The dry climate and the accompanying need for watering lawns and gardens results in a relatively high yearly average per capita water consumption of 1090 litres/day.

This paper highlights the Town's water and sewage treatment control systems. The reasons for integration of the two systems are discussed and the system architecture is outlined. Design and operating features of the new water treatment facility are highlighted. In addition, the control system link with the existing sewage treatment plant (STP) is discussed with the focus on a unique electric load shedding scheme.

BACKGROUND

Prior to the upgrade, the water system consisted of an intake structure on Lee Creek, a pumping station and pipeline to two raw water storage cells with a combined capacity of 295 million litres (ML), and a high lift pump station referred to as Pump Station No. 2 which lifted the treated water up to a 4.5 ML treated water storage reservoir. From the reservoir the water flowed by gravity into the distribution system.

Treatment of the supply consisted of graded gravel medium around the intake pipes at Lee Creek, average of 4 weeks of settling afforded by the raw water storage reservoirs, fine gravel filtration at the outlet of the raw water cells, and disinfection with chlorine gas at the high lift pumps.

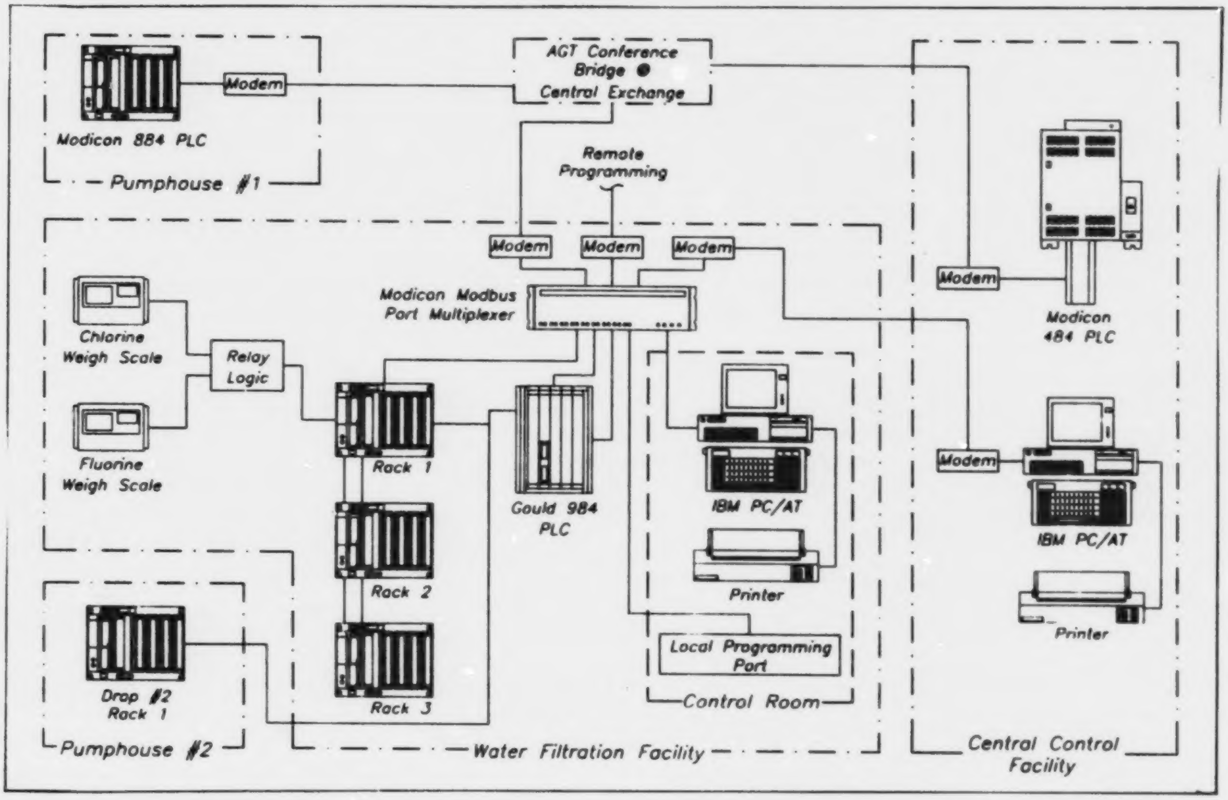
The system required little attention. The operator would check the facility on a daily basis to see if everything was running correctly and to change chlorine cylinders if needed. His visits required less than an hour most days. These part time duties were carried out by the operators of the sewage treatment plant which is manned 8 hours per day.

In considering the addition of the proposed direct filtration component to their water system, the Town did not want to expand staffing.

CONTROL SYSTEMS ARCHITECTURE

To meet the objective of no increase in operations personnel, the new water plant monitoring and control system had to be integrated with the existing sewage plant control system. Figure No. 1 illustrates the outcome of the integration process.

FIGURE 1
Town of Cardston
Integrated Control System Schematic


Modem
Modicon 884 PLC
Pumphouse #1
AGT Conference Bridge @ Central Exchange
Remote Programming
Modem
Modem
Modem
Modicon Modbus Port Multiplexer
Chlorine Weigh Scale
Relay Logic
Fluorine Weigh Scale
Rack 1
Rack 2
Rack 3
Gould 984 PLC
IBM PC/AT
Printer
Local Programming Port
Control Room
Drop #2 Rack 1
Pumphouse #2
Water Filtration Facility
Modem
Modicon 484 PLC
Modem
IBM PC/AT
Printer
Central Control Facility

The first step was to standardize on the type of programmable logic controller (PLC) to be used throughout the water and wastewater facilities. Two Modicon 484 programmable controllers had been installed in the sewage treatment plant which was built in 1983. One is located in the main control panel and the second operates the sludge dewatering filter belt press. These controllers had performed to expectations, so the choice was not difficult.

The water treatment upgrade called for remote monitoring and control of the Lee Creek Pumphouse. A Modicon 884 was selected for that application. A larger Modicon 984 was selected for the control of the water plant. Pumphouse No. 2 is also controlled by the 984 PLC via remote drop No. 2.

The second step was to network the controllers so that the operator had access to the water plant from his offices located across town in the sewage plant. This was done by way of 4 wire leased telephone lines. Modems running at 1200 baud were installed at the water plant, Lee Creek Pumphouse and the sewage plant. The lines were all linked to a communications bridge located in the telephone exchange permitting the use of Modbus, a proprietary communications protocol by Modicon. It uses a master/slave polling routine where the master always initiates the exchange of information and the slave devices can only respond to a query from a master. Usually a Modbus network has only one host or master computer and can have as many as 254 slave

PLC's. However, in this situation, three masters were required. The large Modicon 984 PLC acts as a master because it functions in a supervisory manner over the 884 and the 484 PLC. Two IBM-PC/AT based operator consoles, one each located in the water and sewage plants, also act as masters. A Modicon Modbus multiplexer located at the water plant coordinates all communications. It resolves all conflicts resulting from having three masters by allowing only one to take mastership at a time.

The water treatment plant operates using PLC technology for the control functions and IBM-PC/AT based software package called "Process Windows" for operator interface with the system. Control of the valves and pumps, and the monitoring for alarm conditions remains the function of the PLC. The traditional control panel, however, has been replaced with the personal computer, colour monitor, and printer.

The switch to the PC based interface is significant. The virtue of the traditional control panel is its simplicity; it consisted of a steel box with a number of buttons, lights, switches,and charts mounted on the front panel. However, its simplicity was also a very severe liability. Control panels are hard-wired which makes them inflexible if there are modifications or changes to the system. They are costly to build. Most significantly, they are typically uninformative about the process. Since this is the operator's primary window into the control system, this is a major drawback in optimizing (or even maintaining) control of the process.

The personal computer replaces the control panel with graphic representations of the process in sufficient detail that inexperienced users can readily recognize the various systems within the plant. All significant process points in the plant are monitored and the information is displayed on one or more of the process windows.

PROCESS WINDOWS

A software package called "Process Windows" by Taylor Industrial Software Ltd. is in use at the Cardston facilities to provide the man-machine interface. The software makes use of colour graphics screens called process windows that are configured by the control system programmer to display status and alarm conditions in both discrete (e.g. on/off, flashing colours) and analog (e.g. numbers, barcharts) form.

Two "master station" IBM-PC/AT's are in use; one at the water treatment plant and one at the sewage treatment plant. The master station receives and displays all process information from the PLC which, in turn, is collecting data from the field sensors and control elements in the plant. The information displayed by the master station is updated continuously and any alarms conditions are annunciated both visually and audibly.

The master stations at the WTF and STP are linked by telephone line. From either, the operator can monitor and make process changes to the WTP. Alarms will be reported and can be acknowledged from both master stations. At present the link to the STP is primarily for alarm reporting and for the load-shedding utility discussed below. No control functions for the STP have been incorporated into the graphics system; however, this is a matter of configuring the process windows that have been reserved for this purpose.

In addition to the real-time operating and reporting functions, the graphics system also generates the bulk of the production reports required for operating the WTP. A daily report is automatically generated and additional reports can be assembled at other time intervals as required. The reporting process starts with the Process Windows operating system writing plant data to a spreadsheet file. The Lotus Symphony integrated software package is used to retrieve and format the data.

The virtues of a graphics system and its link to the STP are in the ease of operation and savings of manpower. As noted above, the STP is operational headquarters for the town. The commissioning of the water treatment plant has not changed that since the graphics system provides all necessary monitoring and alarm capabilities which eliminates much time and travel between the WTP and STP. The only regular manpower requirements at the WTP are for about two hours per day for sample collection and testing, checking on the chemical supplies, and sweeping the floors.

WATER TREATMENT PLANT

The upgrade of the Town of Cardston's water system involved adding flocculation and direct filtration components housed in a 468 square metre building located between the Pumphouse No. 2 and the clearwater reservoir. A second 4.5 ML clearwater reservoir was added at the same time. The raw water quality is relatively good, so no sedimentation facility was provided, although this can be added at a later time if required. The nominal capacity of the plant is 17.28 ML/day to serve a 20 year projected population of 5,175 people.

Two sources supply the raw water reservoirs: the primary source is from Lee Creek via pipe inlet and pumpstation; and the secondary source is the Leavitt Irrigation District canal via a gravity pipeline. When in use the raw water reservoirs are topped up automatically by having the controller in the water plant signal the controller in the Lee Creek pumphouse to start and stop the pumps. The delivery from the irrigation canal is relatively new and was implemented to augment low flows in Lee Creek during late summer and fall as experienced in 1984 and 1985. It is operated manually by setting a hand valve.

The water treatment facility is operated as a batch process. When the level of treated water in the clearwater reservoirs falls to a preset value the process starts and treats water at a fixed rate until the reservoirs are topped up.

The existing pumpstation No. 2 pumps water from the raw water reservoirs into the filtration plant at a preset rate by the influent flow control loop. Alum (and optionally polymer as a coagulation aid) along with a low dosage of chlorine are mixed into the water using a static mixer.

The water is passed through 6 hydrodynamic flocculation cells arranged in three banks of 2 cells. As the water gently spirals upward through the cells a pin head sized floc is formed. Flocculated water flows into a filter influent channel where it splits into four streams and passes through the dual media declining rate filters (media are anthracite and sand).

Flowrates for the individual filters are not controlled. The total throughput is set by maintaining a constant level in the filter influent channel. A common output signal from the level control loop is fed to all four filter effluent valves. The flow through each filter becomes a function of the head loss across the media.

A single turbidity analyzer monitors filter effluent on a sampling basis under control of the PLC. High effluent turbidity from a filter will create an alarm and cause the filter to go off line.

Chlorine and fluorine injection take place in the filter effluent header as the water flows to the highlift pumpwell. Digital weigh scales monitor the amount of chemical used and relay the information to the PLC.

Highlift pumping is accomplished using three variable speed pumps. The pumps are staged on and off and their speed is regulated to maintain a constant level in the pumpwell.

Backwashing of the four filters is completely automatic, initiated by either filter run time or the volume treated. In lieu of having separate dedicated backwash pumps, this function is accomplished with the high lift pumps. When a backwash is about to occur, the role of the highlift pumps is switched to backwash mode by closing a block valve in the line leading up to the clearwater reservoirs. Using the two larger highlift pumps, the backwash flow is set, by adjusting pump speed.

The highlift pumpwell level control is the most complex in the plant. When not backwashing, the level is controlled using the highlift pumps to balance outflow to inflow. During backwash, the level is maintained by supplementing plant production with water from the clearwater reservoirs. This is done because the highlift pumpwell has insufficient volume for backwashing. To overcome this, the PLC controls the level by manipulating a valve in a supply line from the clearwater reservoirs.

After backwashing, the waste is routed to the backwash effluent ponds where it is eventually decanted back into the raw water reservoirs.

The elimination of backwash pumps and their concomitant large wetwell and the automated control of the backwash function using the high lift pumps resulted in a saving of 10% to 20% of the total floor area of the plant.

SEWAGE TREATMENT PLANT

The Town's existing sewage treatment plant is a secondary treatment facility that has been in operation for about 6 years. The plant is automated using PLC technology and traditional control panel man-machine interfaces. In addition, sufficient shop and office space has been incorporated into the facility to essentially meet all the needs of the Town for both its sewage and water treatment plants. This permitted a more spartan and less costly design at the water plant.

As such, the sewage treatment plant remains the headquarters for municipal operations. It was by direct request of the Town administration that the control system for the water plant be linked with the sewage treatment plant so that a minimum of manpower would be expended in being at, and travelling back and forth to, the water plant for routine operations.

ELECTRIC LOAD SHEDDING

Electric load shedding is an additional facet to this project that was incorporated into the control system. The Town acts as a utility in that it owns the local distribution system and purchases power in bulk from the generating utility. As a cost saving measure, the Town had instituted a load-shedding program to clip the peak demands and thereby avoid high demand charges from its supplier. The town also extended its electric load-shedding program to include the WTP and the Lee Creek pumpstation, along with the STP and the Town ice arena.

Power is metered at the custody transfer point and input to the PLC at the STP. The PLC performs the logic for dropping and picking up the loads according to priorities established by the Town. By reducing the overall demand, the Town saves $20,000 to $30,000 per year on its total electricity bill.

SUMMARY

The Town of Cardston's new water treatment plant is a fully automated facility requiring minimal manpower for operation and supervision. Programmable controllers provide flexible reliable control capable of performing routinely plant operations that are difficult to undertake manually.

The plant's control system is linked with the existing sewage treatment plant which acts as headquarters for municipal operations. The control system uses colour graphics to display process conditions and alarms and, as well, it generates the bulk of the production reports required for the water treatment plant. Significant savings in manpower are realized with this centralized operation of both the water and sewage plants.

Electric load-shedding has been implemented with savings on the purchased power in the range of $20,000 to 30,000 per year.

RURAL MUNICIPAL WATER SUPPLY
MULTI-USER NEEDS / MULTI-FUNDING APPROACHES

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In Alberta, rural residents in small hamlets and on farms are expecting better and more reliable potable water quality as well as increased availability of water for agricultural use. As a result, local and senior governments are having to provide funding assistance and project guidance to meet these expectations.

MUNICIPAL WATER SUPPLY

In the past 2-3 decades, the government of Alberta has been very active in regulating and funding the potable water supply systems (and wastewater treatment systems) of municipalities in the province. Through the Public Health Act and the Clean Water Act and other pieces of legislation, standards for public water supply systems have been instituted and enforced. These standards have been sufficiently stringent that the capital costs of the supply, treatment and distribution systems required in the municipalities have been financially assisted by the provincial government. Project funding assistance has ranged from 50% - 75% of the initial capital cost and in some cases even higher percentages have occurred. As well, operating assistance is provided in the form of operator training and certification programs and technical advice services.

This approach has been reasonably effective in ensuring adequate public water supply standards for incorporated municipalities with populations of 300-500 people and more.

In unincorporated municipalities, i.e. hamlets within the boundaries of Municipal Districts, Counties and Improvement Districts, the same programs have been applied. These communities of 100 people or less, relative to larger communities, frequently suffer from large per capita project costs due to economies of scale.

RURAL WATER SUPPLY

On farms and acreages in Alberta, water supply has historically been left in the hands of the private landowner. Water quality standards of the private user are not as stringently regulated as public water quality. This has allowed the large majority of rural residents to use groundwater wells for their domestic water needs. These wells are frequently drawing from aquifers that produce water exceeding the maximum public water quality standards. Where the water quality is especially poor, trucked in water frequently is used to supplement water supplies for cooking, drinking and washing needs.

The government of Alberta has financial programs to assist rural residents in development of groundwater wells for their homes. In areas where groundwater quality is poor however, this is of limited benefit to the resident for providing good quality water.

In recognition of this fact, in recent years the Province of Alberta and the Federal Government (through PFRA) have established a funding assistance program for providing water supply systems to "rural water cooperatives". These cooperatives can consist of 15-50 farmers served by small diameter water distribution lines emanating from a "good quality" water source.

RURAL WATER REQUIREMENTS AND USER NEEDS

In urban municipalities treated water is provided for domestic, commercial and industrial use as well as fire fighting purposes. The standards of quality are heavily dictated by the health and safety concerns of the users who physically consume it. Where industries require different water quality characteristics for process or cooling needs, they are generally required to install their own supply and/or treatment system to obtain the special chemical characteristics that are required.

Increasing concern about the quality of urban municipal water due to pollution of raw water sources and a wider awareness of potentially harmful chemicals has resulted in ever more stringent drinking water standards. These urban standards are legislated across the province and are demanded by the rural consumer when a public system is provided.

In localities where raw water sources have a history of poor water quality and quantity characteristics, the costs of providing a water supply and treatment system to meet drinking water standards are significant. Not only can treatment be costly, but the per capita costs are very high because of the relatively small number of water consumers. Also, the need for good water for residents in a "poor water" rural area can be greater in non-domestic use than in better served areas. As well as individual domestic use, acceptable water is needed for:

- fire fighting
- livestock watering
- irrigation
- chemical spraying
- institutional servicing

CASE HISTORIES

Two examples of current rural community servicing in Central Alberta are described in the following. These illustrate the present approach to solving water quality servicing in poor raw water/low population density areas. Engineering, project funding and operational cost concerns are reviewed.

Pickardville Water Supply Project

The hamlet of Pickardville is located in the Municipal District of Westlock, approximately 100 km north of Edmonton. This community of 300 people was served by groundwater wells that were experiencing reduced flow capacities and poor chemical characteristics. In 1986 possible long term solutions to the community water supply problems were examined.

Groundwater sources in the area were found to have low volumes and poor quality. Surface water sources in the area also provided inadequate long term volumes.

The hamlet is located approximately 12 km south of the Town of Westlock. The most feasible solution to Pickardville's problem was determined to be the installation of a treated water pipeline from the Town's system to the hamlet and the establishment of a long term water purchasing agreement between the M.D. of Westlock and the Town of Westlock.

By 1987 a 12 km pipeline, 150 mm in diameter, had been installed between the two communities complete with metering facilities, a booster pumping station and distribution system improvements at both ends. Because the pipeline traversed an area of generally poor water supply, local farmers and businesses along the route were provided with water services as well.

The capital cost of the project was approximately $600,000. The Province of Alberta, through Alberta Transportation and Utilities, funded 75% of the costs. The Federal Government, through PFRA, funded 15% and the balance of the capital costs were paid for by the Municipal District. The M.D. of Westlock pays 100% of the system operating and maintenance costs and charges consumers for use of the water.

Neerlandia Water System

The hamlet of Neerlandia is located in the County of Barrhead approximately 150 km northwest of Edmonton. This community is centred in an intensive farming area north of the Town of Barrhead and the entire local area is subject to poor water quality problems.

The community has a population of approximately 100 people. Two large churches, a County school and a large farm supply/service centre are located in the hamlet. The entire hamlet was served by individual shallow water wells. In 1988 the County decided to provide Neerlandia with a public water supply system.

The most feasible water supply option was to utilize a nearby shallow surface fed lake as a raw water source and construct a central conventional water treatment plant (flocculation/sedimentation/filtration) in the hamlet.

Several factors complicated the sizing and design of system components. As the central source of quality water in the area, many users in and around Neerlandia wanted access to the system. The relatively large buildings in the community necessitated large reservoir capacities for fire fighting purposes. Area farmers wanted to truck out treated water to supplement their domestic sources. As well, untreated water was needed to assist them in operating their livestock operations. Balancing the water demand of these differing and sometimes conflicting consumer needs was costly and in some ways inefficient.

In 1989 the system was in place. The capital cost of the project was approximately $800,000. The Province of Alberta, through Alberta Transportation and Utilities, funded 75% of the costs. The Federal Government, through PFRA, funded 12% of the costs, and the County of Barrhead picked up the balance. Operation and maintenance of the system is paid by the County and consumers pay for their use of the water from locally applied user fees.

COMMENTS AND DISCUSSION

1. Drinking water quality standards are becoming increasingly more stringent and this trend is expected to continue. The costs of equipment and materials to provide the necessary levels of water quality are increasing accordingly. In larger urban centres which experience a large service population and a reasonable raw water quality, these increased costs may be tolerable. In rural areas with less benefitting consumers, the increased costs may put the expense of properly treated water out of reach of the rural consumer.

2. To achieve the current water quality standards, treatment equipment is becoming increasingly complex. In rural and remote areas, this complexity can be a strain on the capabilities of the trained operator. As well, maintenance of the equipment can be difficult. Breakdowns and parts replacement can be costly and time consuming.

3. Central treated water supply plants in areas where general water quality is poor should incorporate the supply needs of a non-domestic nature where possible. The availability of nonchlorinated water and raw water for specialized use and bulk use should be considered during the conceptual design of the project.

In summary, in the coming years more and more rural and remote areas will be demanding properly treated water. Current legislation will force them to use expensive water treatment methods. Consumers in these regions need better water quality for more than just domestic use.

A coordinated assessment by government agencies, local governments and consumers should be made to realistically combine the needs of the rural water user with the demands imposed by the legislated water quality standards.

URBAN INFRASTRUCTURE UPGRADING
A CASE STUDY

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GCG Dillon Consulting Limited

INTRODUCTION

Deterioration of urban infrastructure has become a widely recognized concern during recent years. In adequate and sometimes inappropriate expenditure on maintenance and upgrading of existing infrastructure has resulted in increasingly frequent failures. Such failures often cause considerable public disruption and inconvenience, as well as property damage and personal injury. In addition, the cost of remedial work is often disproportionately high when compared with properly planned maintenance or replacement. Among the more common examples of failure resulting from infrastructure deterioration are watermain breaks and sewer flooding. These are typical of the problems that occur due to the ageing of our urban infrastructure, accompanied by inevitable structural deterioration and increasingly serious capacity shortfall.

Because the traditional levels of expenditure on maintenance have not proved adequate to cope with the problem of infrastructure deterioration, most urban municipalities now realize that much larger amounts will have to be spent in the near future if serious breakdown of municipal services is to be avoided. Once the need for action is accepted, the municipality faces a number of problems. These include:

- The difficulty of allocating tax dollars to infrastructure upgrading when this may not yet be viewed by the public as a serious problem.
- The negative public response when popular programs or works are cut in favour of infrastructure upgrading.
- The difficulty of obtaining funding from Provincial and Federal Government sources.
- The problem of deciding where to start, when faced with widespread infrastructure deterioration.
- The decision as to whether in-house or private sector engineering and construction resources should be used.

This paper outlines a major problem of infrastructure upgrading undertaken by the City of Edmonton and describes the work completed to date in the Oliver Neighbourhood.

PROGRAM OVERVIEW

In 1986 the City of Edmonton was becoming increasingly aware of the escalating costs of maintaining the City's older infrastructure. The number of watermain breaks had increased from 100 in 1590 to 2000 in 1985 and an increase in the frequency of surface and basement flooding was apparent in some areas. In some neighbourhoods the condition of sidewalk and roads was poor and, in the central area of the City, bare steel gas lines were causing problems.

In 1986 the Alberta Provincial Government announced an unconditional per capita grant to municipalities to stimulate employment in the fields of public works engineering and construction. This program is known as AMPLE (Alberta Municipal Partnership in Local Employment). The City of Edmonton decided to utilize most of the funds to be received under this program (120 million dollars over eight years), for infrastructure upgrading in several older City neighbourhoods. The City's AMPLE Neighbourhood Program currently involves infrastructure upgrading work in the neighbourhoods of Oliver, Westmount, Inglewood and Britannia-Youngstown.

In the spirit of the objectives of the AMPLE grant program, the City elected to have the engineering and construction carried out by private consultants and contractors. Overall control and direction is provided by a relatively small simple management structure comprising senior City staff. GCG Dillon Consulting Limited was appointed to carry out the engineering work for the Oliver Neighbourhood.

The specific objectives of the City's AMPLE Neighbourhood Program are:

1. Upgrade sewers, watermains and roads, allowing where appropriate for coordination with planned work on other services such as gas, power and telephone.

2. Create employment.

3. Construct community desired amenities within the roads right-of-way, in conjunction with the infrastructure work.

4. Utilize private sector resources.

OLIVER NEIGHBOURHOOD

The Oliver Neighbourhood covers a 60 square block area straddling Jasper Avenue immediately west of downtown Edmonton. The residential population of approximately 16,000 is accommodated mainly in walk-up and high rise apartments with single family homes making up the balance. The neighbourhood also contains extensive commercial development as well as a hospital, churches, schools and community league facilities.

Although development of Oliver began in the late nineteenth century, the infrastructure was constructed after the turn of the century. The neighbourhood is serviced by a combined sewer system totalling 30 kilometers in length. Prior to 1950 most of the sewers were constructed of tile pipe and these sewers are now in generally poor condition. The water distribution contains 20 kilometers of mains, 150 mm and 200 mm in diameter. Most of the mains are cast iron and some deterioration due to external corrosion has occurred. The road system comprises 18 kilometers of streets. These vary in condition from poor to good, as do the lanes and sidewalks. Streetscape amenities are generally limited to trees and bench seats.

PROJECT OUTLINE

In order to meet the City's overall program objectives the following project outline was developed for the work in Oliver.

Phase 1

1. Identify deficiencies in existing watermains, sewers, roads, lanes and sidewalks.
2. Identify community amenities desired by Oliver residents. (Limited to amenities within road rights-of-way, i.e. streetscape amenities).
3. Establish priority list of all upgrading work.
4. Identify the infrastructure upgrading and streetscape work to be carried out over a three year period.
5. Inform the community of progress and obtain community input on the streetscape improvements.

Phase 2

1. Establish a detailed construction program for each year, based on the priority listing from Phase 1.
2. Design and tender the work in suitable sized contracts.
3. Carry out construction within the allocated time and budget, with minimum public disruption.
4. Continue the community information/input process during construction.

DEFICIENCY IDENTIFICATION

Link Identification

In order to manipulate and analyze the large amount of data produced, it was necessary to develop a system to uniquely identify every individual infrastructure component. To achieve this the Oliver area was defined as a network of links, each link comprising of a section of roadway, laneway, or right-of-way, running from one intersection to the next intersection. Watermains were identified from valve to valve or tee to tee, within each roadway or laneway link. Sewers were identified from manhole to manhole within road and lane links.

Database

A database was developed for each of the major components (watermains, sewers, roads). This system was designed to contain physical descriptions, performance characteristics, and importance ratings for each link of each of the three major components. With every component in the system assigned to a particular link, the three databases are inter-related and every component of the system can be assessed through the link system.

Watermains

Deficiency identification for watermains involved assessment of structural condition and hydraulic adequacy. Structural condition was assessed by a review of main break records. Each watermain link was rated for structural performance on a scale of 1 to 5 with 5 representing the worst condition. Level 5 on this scale represents a record of 5 breaks per year per kilometre. Hydraulic performance was assessed by means of a computer simulation program. This program determined the pressure at each node and the flow through each link for a number of different demand conditions. The performance of each link was rated on a scale of 1-3, with 3 representing the worst performance.

On completion of the structural condition assessment it was apparent that most of the mains in Oliver are in good condition with 90% rated 1 and only 1% rated 5.

The hydraulic performance computer model was run for peak hour demand condition and maximum day plus fire condition. Actual consumption values for major users such as the General Hospital and Molson's Brewery were used. Frictional resistance values (C-values) were determined by conducting hydrant flow tests. Mains of various ages and diameters were chosen and the results indicated average C-values ranging from 49 to 72. The results of the modelling indicated that the water distribution system in Oliver generally performs well, with very few links exhibiting low pressure or high head losses. THis high standard of hydraulic performance is due to the presence of several large diameter transmission mains in the neighbourhood and the close proximity of the Rossdale Water Treatment Plant.

In addition to structural and hydraulic assessments, the water system was checked for adequacy of fire hydrant coverage and minimum main diameter.

Sewers

Identification of sewer deficiencies also requires an assessment of both structural and hydraulic characteristics. All accessible sewers in Oliver were inspected by closed circuit television camera (CCTV). The videos, still photographs and record logs were then examined and each sewer link rated for structural condition on a scale of 1 to 5. In order to achieve consistent results when viewing large numbers of defects, a rating system developed in the United Kingdom based on colour photographs of typical defects was used. The ratings indicate different levels of risk of failure, with 5 indicating imminent or actual collapse of the pipe. Approximately 11% of the sewers inspected were rated 5. The remainder were fairly evenly distributed over the range of structural performance grades 1 to 4. Where localized structural defects not typical of the whole link were observed, these locations were recorded as requiring spot repairs.

A computer simulation was carried out by the City of Edmonton to determine surcharge levels at each sewer link for a 1 in 5 year design storm. The degree of surcharging enabled an assessment to be made of the likelihood of basement flooding, surface flooding, and overflow to the North Saskatchewan River. Hydraulic performance ratings from 1 to 3 were assigned to each link. Level 3 represents a surcharge to at least 1.2 metres below ground level. Most of the sewers (88%) were found to be hydraulically satisfactory. However of the remaining 12%, most were rated 3.

Roads

Deficiency identification for roads was broken down into four sub headings:

- pavements
- lanes
- curbs and gutters
- sidewalks

Pavements were assessed using the American Public Works Association Paver Method (APWA). This method requires a visual examination of the pavement on the basis of various elements of pavement distress. A structural performance grade of 1 to 5 was allocated to each link of pavement, with 5 representing a failed condition.

Lanes were also evaluated by the APWA method and the same rating scale was used.

Curb and gutter was visually examined and a condition rating of 1 to 5 allocated to each link.

Sidewalks were evaluated in the same way as the curb and gutter. On average, the pavements in Oliver are in worse condition than the lanes, sidewalks and curb and gutter. Approximately 25% of the pavements were rated 5 compared to very small percentages for the other components.

INFRASTRUCTURE PRIORIZATION

As it was not possible to upgrade all decayed infrastructure within the time and budget available, an integrated priority rating reflecting the condition and importance of each component was prepared for every link. Figure 1 illustrates the way in which this information was presented.

The boxes contain overall condition ratings for sewers, watermains and roadways. These overall condition ratings reflect the structural and hydraulic condition ratings already described, as well as a sensitivity (importance) rating. The sensitivity ratings indicate the relative consequences of failure. Sewer sensitivity was rated 1 to 3 with 3 representing the most serious failure consequences. Sewers were rated for sensitivity on the basis of depth, diameter, location under major roads, railways and buildings, etc. A sensitivity 3 sewer could, for example, be of large diameter, over 6 meters deep, under an arterial road, and be likely to cause disruption to hospital access in the event of failure. Watermains were rated for sensitivity in a similar way with depth, diameter, population serviced, service to hospitals, etc. as rating criteria. Pavements were rated on the basis of average daily traffic numbers, with an adjustment to allow for truck routes and bus routes. Lanes were rated on the basis of type of construction and presence of underground utilities. Sidewalk ratings reflect the amount of pedestrian traffic.

Curb and gutter was given the same sensitivity ratings as the pavements. Pavement, curb and gutter and sidewalk sensitivity ratings were combined with the respective structural performance ratings to provide overall condition ratings for these components. These overall condition ratings were then combined, resulting in a roadway overall condition rating. Separate overall condition ratings were calculated for lanes. The overall condition ratings for sewers and watermains obtained by combining the structural and hydraulic performance ratings with the sensitivity ratings.

Where appropriate the constituent performance and sensitivity ratings were weighed to realistically reflect their relative importance in the overall condition rating.

The need for rehabilitation of a water, sewer or road link is indicated by an asterisk beside the overall condition rating. The need for rehabilitation was determined on the basis of the following criteria:

- Sewers rated SPG 5.
- Sewers identified as hydraulically inadequate in City of Edmonton's storm relief program.
- Watermains rated SPG 2 to 5.
- All watermains located in lanes adjacent to sewer replacement.

- All roads and lanes requiring repaving because of sewer, water or gas replacement.
- Roads (not lanes) rated SPG 5.
- Some roadways rated SPG 4.

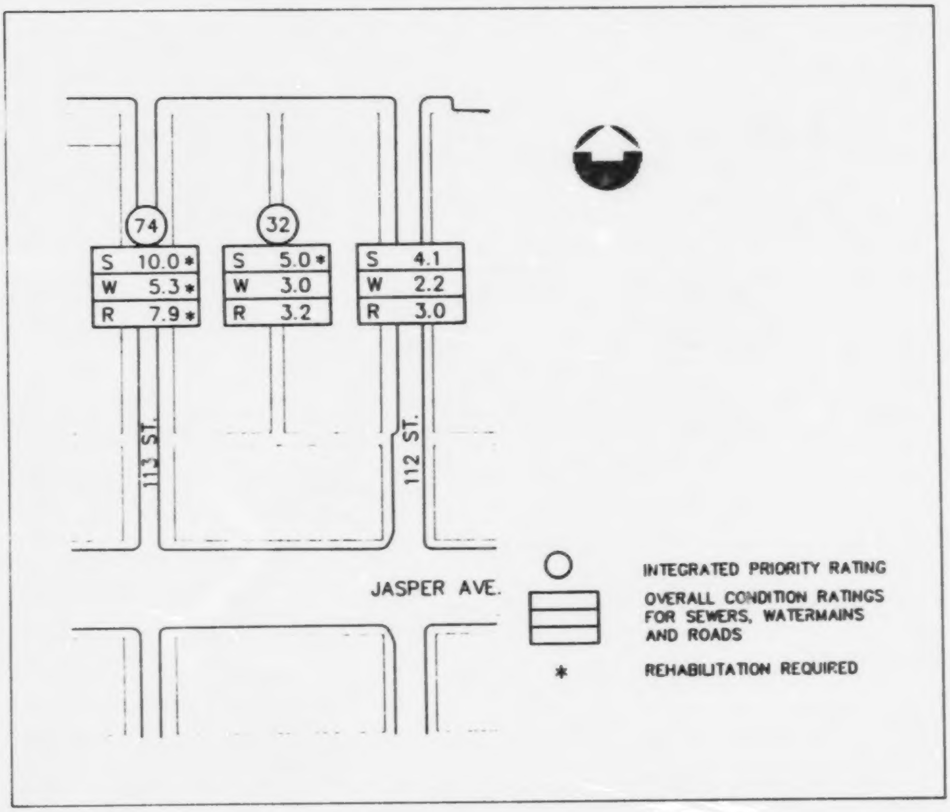
It can be seen that the decision to carry out upgrading work in a particular link could be made entirely on the basis of the above criteria without the need to develop overall condition ratings and integrated priority ratings. However, this approach could lead to upgrading of individual infrastructure components without regard for the overall importance of the component. For example 150 mm watermain rated SPG 5 should have a lower priority than a 300 mm watermain with the same structural performance grade. The overall condition rating incorporating a sensitivity factor facilitates these decisions.

The integrated priority rating, shown circled in Figure 1, indicates the overall need for upgrading work on all components of a link. This rating provides a means of avoiding the situation where a road in good condition is dug up to replace a small low sensitivity sewer.

The integrated priority rating also includes a weighing to allow for planned upgrading of utilities outside the scope of the AMPLE program. These include gas, power, telephone and cable TV.

FIGURE 1

Integrated Priority Ratings


74
32
S 10.0*
W 5.3*
R 7.9*
S 5.0*
W 3.0
R 3.2
S 4.1
W 2.2
R 3.0
113 ST.
112 ST.
JASPER AVE.
INTEGRATED PRIORITY RATING
OVERALL CONDITION RATINGS FOR SEWERS, WATERMAINS AND ROADS
*
REHABILITATION REQUIRED

Provision of the information shown on Figure 1 for all links in the project area, creates a sound basis for developing a construction program to fit any desired time period and budget.

STREETSCAPE AMENITY PRIORIZATION

A major part of the project was to identify deficiencies in the streetscape features of the Oliver community, and to develop an improvement program, consistent with the desires of the community, for construction in conjunction with the infrastructure upgrading.

Existing streetscape amenities are limited to grass boulevards, trees, street lighting and bench seats. Since existing mature trees provide on of the Oliver's major streetscape assets, a tree planting program to replace damaged and missing trees was established.

Consultation with the community was and continues to be a major feature of the AMPLE Neighbourhood Program. This consultation process was utilized by way of a series of community meetings and workshops with the objective of compiling and priorizing a list of potential streetscape improvements desired by the community. This process was strongly supported by the Oliver Community League and the Community of Oliver Group. A questionnaire-newsletter was delivered to all residences to test the validity of the input from the two community groups. The potential improvements considered by the community included those already identified in the Oliver Area Redevelopment Plan.

This community consultation process was very successful and resulted in a comprehensive list of potential streetscape projects, priorized in order of importance to the community.

THREE YEAR INFRASTRUCTURE AND STREETSCAPE PROGRAM

The priorized listing of infrastructure upgrading works was presented to the City in the form of a Recommended Rehabilitation Plan, complete with cost estimates. On the basis of this plan and known budget constraints, the City suggested a three year construction program. This program was estimated to cost approximately 14 million dollars.

From the priorized list of potential streetscape projects, three major projects and a number of small isolated improvements were recommended. The major projects involved comprehensive improvements along 100 Avenue, including extension of the Heritage Trail System and improvements at selected sites on 102 Avenue and 111 Street. Due to the conceptual nature of these projects, it was realized that further design refinement and community discussion would be needed before reliable cost estimates could be prepared. For this reason the three year construction program did not include the cost of the recommended streetscape works.

The results of the work carried out in Phase I were submitted to the City of Edmonton in a comprehensive report in the fall of 1987.

DETAILED CONSTRUCTION PROGRAMS

Based on the three year construction program established in 1987 detailed programs were prepared for 1987, 1988 and 1989. These programs incorporated some additions and deletions to the original three year program, as well as updated cost estimates. Each years' work was divided into a number of contracts whose size, contents, and locations were established with the following objectives in mind:

1. Work to be carried out in order of urgency as indicated by priority ratings.

2. Work to be concentrated in one area of the neighbourhood to avoid repeated disruption during successive years.

3. Size of contracts to be optimized to provide work for as many contractors as possible without creating site overlaps and coordination problems.

The streetscape works to be undertaken in each year were established after continued community consultation, preliminary design, and estimation of costs. These works were designed for inclusion in the infrastructure construction contracts. Tree replacements was arranged in several separate contracts, located and placed so as to avoid damage by the other works.

DESIGN AND TENDERING

Design of the infrastructure works involved several problems not normally encountered in new construction. These included:

1. The requirement to match existing sewer elevations while attempting to provide increased capacity in specific links.

2. Restricted construction areas in lanes and streets where total closures were not acceptable.

3. The presence of many existing utilities both above and below ground.

4. The need to attempt to provide road drainage grades to current City standards while constrained by the grades of existing features such as intersecting streets, sidewalks, property lines and tree bases.

5. The requirement to minimize disruption to parking, traffic movement and pedestrian access.

6. The difficulty of testing replacement sewers and watermains while minimizing service interruptions and, in the case of watermains, avoiding damage to existing services and plumbing during pressure tests.

Some of these problems were addressed during the design process. For example, the traditional requirement for exfiltration texting of sewers was eliminated and replaced by a mandatory CCTV inspections. After some serious problems during the first contracts, traffic disruptions were reduced, if not eliminated, by specifying scheduling requirements that avoided road closures in adjacent streets and lanes. Discussions with City personnel resulted in relaxation of some minimum road grade requirements in recognition of the unique problems faced when upgrading existing city streets. Where conditions were suitable, the problems of restricted working space and disruption to traffic, pedestrians and existing utilities were avoided by specifying the use of no-dig rehabilitation of sewers and manholes. The problems associated with testing and disinfecting replacement watermains were recognized by City staff who responded by writing amended specifications tailored to address the problems encountered in replacement works.

In view of the success of the community consultation process in Phase I, it was considered essential to continue to keep the community informed during the design and construction stages. This was achieved by means of further public meetings and newsletters. By this means the community was kept informed of the proposed works and given advanced notice of the likely construction impacts. In addition, special clauses were included in the contract documents requiring the contractors to provide a reasonable level of assistance to those affected by the construction work. These requirements included written notification to all residents and businesses of construction start dates, durations and expected impacts. The contractors were also required to arrange alternative parking, access and utility service where these were to be interrupted for other than brief periods.

A coordinated tendering schedule was prepared by the City to ensure that tenders for all AMPLE contracts closed at different times. This provided maximum opportunity for interested contractors to bid these works. With a few exceptions, tenders were called from January through to May of each year. This generally resulted in early construction starts with most contracts achieving construction completion within the same year.

CONSTRUCTION

To date eight contracts totalling 8.9 million dollars in value have been awarded for work in Oliver. The tender amounts and actual final costs for those contracts completed to date are shown in Figure 2. Experience with these contracts has shown that, in spite of the unique problems faced in infrastructure upgrading work, good cost control can be achieved by a combination of clear but comprehensive plans and specifications and diligent construction inspection.

FIGURE 2
Tender Prices vs. Actual Final Costs

CONTRACT NO.	TENDER PRICE	FINAL COST
0.1	$1,449,035	$1,331,878
0.2	$ 933,894	$ 898,323
0.3	$ 657,379	$ 643,895
0.4	$1,000,257	$1,091,864
0.5	$ 567,560	$ 624,832
Tree Planting	$ 17,094	$ 17,514
TOTAL	$4,625,219	$4,608,306

A brief summary of the work completed at the end of 1988 is shown in Figure 3. As would be expected, unit costs for infrastructure upgrading are generally higher than for equivalent new construction.

FIGURE 3

Construction Quantities for 1987-88

Watermains	3,300 m
Hydrants	15
Valves	100
Sewers (conventional)	3,500 m
Sewers (In-situ renovation)	150 m
Manholes	70
Curb and gutter	2,300 m
Sidewalk and Aprons	3,700 m^2
Pavement - lanes	11,000 m^2
- roads	16,000 m^2
Trees	55

Plus associated fittings, service connections, catchbasins, surface restoration, etc.

Figure 4 provides a few examples of unit costs from Oliver contracts. Pre-tender cost estimates have generally been higher than the low bids received. It was thought that after the first full year of construction on the AMPLE Program, the many unexpected problems encountered by the contractors would force prices up in the following years. This has not happened and most contracts continue to attract good numbers of relatively close bids.

The construction work to date has not been without its problems. Some of these were discussed in the previous section. Additional problems encountered include:

1. Coordination with the Northwestern Utilities Ltd. bare mains replacement program. This program of replacement and relining of an extensive network of gas mains and services within the Oliver area necessitated a coordinated approach to minimize public inconvenience and avoid re-excavation of new pavements. No direct contractual relationship existed between the AMPLE contractors and NUL. Coordination of AMPLE works with the gasmain upgrading was therefore based on voluntary cooperation. Although the respective schedules were coordinated before construction commenced, unavoidable delays in the NUL work resulted in delayed completion of some of the AMPLE works. There have also been occasions when AMPLE construction delays have caused problems for NUL.

2. Public inconvenience. In general, community members were prepared to accept the disruption caused by the works. However when progress at a particular location appeared to the public to be unreasonably slow, complaints were received. Most complaints were received by the Citizen's Action Centre and passed to the City, the contractors, or the consultants. Although the majority of complaints were resolved fairly quickly, this was not always the case. An example of a more difficult problem was the noise generated at a contractors' site office and materials storage area, located in a single family residential area. The noise

created by trucks and loaders (including back-up warning signals) all day, six days a week caused understandable concern among neighbours. This problem was finally resolved by the relation of the contractor's storage operations to a more remote site.

FIGURE 4

Average Unit Costs

WATERMAINS:		
-	replace existing 150 mm with new 150 mm	$272/m
-	replace existing 200 mm with new 200 mm	$310/m
-	replace existing 200 mm with new 450 mm	$416/m
-	replace existing hydrant	$2540 each
SEWERS:		
-	replace existing 300 mm with new 300 mm	$320/m
-	replace existing 375 mm with new 375 mm	$360/m
-	replace existing 375 mm with new 600 mm	$403/m
-	replace existing 1200 mm manhole with new 1200 mm manhole	$3860 each
-	renovate existing 375 mm by Insituform	$472/m
-	renovate existing 500 mm by Insituform	$580/m

During the course of the work constructed to date, several relatively new processes and products have been used. These include:

1. The use of no-dig methods of sewer rehabilitation. The Insituform process has been used to rehabilitate over 1000 metres of sewers in the Oliver neighbourhood. This method places a custom made resin impregnated fabric tube into the sewer by inversion under water pressure. After inversion the thermosetting resin is cured by circulation of hot water within the tube. The result is a strong, impermeable, smooth pipe within a pipe. While this technique has not always reduced direct costs, it provides significant benefits due to virtual elimination of surface and sub-surface disruption. In addition, installation time is generally reduced to 24 hours per block compared to as much as a week for conventional open cut construction. Another no-dig method that has been used to a limited extent to date is the Line-Pipe technique. This system inserts a hinged, folded PVC pipe into an existing sewer pipeline for repair of a local structural defect. Once in position the Link Pipe is expanded by means of an air cell until it locks into a circular shape against the inside of the damaged pipe. This method, and a variation utilizing stainless steel, have proven useful for localized repairs where the depth of the sewer or existing surface features preclude excavation.

2. In-situ rehabilitation of decayed manholes. Special segmental forms are used to facilitate placement of a high strength concrete lining against the existing manhole walls. This method

reduces construction disruption and appears to be significantly less costly than complete manhole replacement.

3. Compaction of trench backfill by means of hydraulic "hoe-pack" units and remote controlled vibrating plate compactors. These improve speed of compaction and improve safety in unstable trench conditions.

CONCLUSION

With one more year of construction remaining to complete the AMPLE Neighbourhood Program in Oliver, a review of the achievements to date indicates that the program is achieving worthwhile results. The work has provided employment in the local private sector engineering and construction industries, the local economy has been benefitted from a considerable cash infusion, cost effective replacement and upgrading of deteriorated infrastructure in the Oliver Neighbourhood is well under way, constructing of unique and imaginative streetscape improvements will be completed this year, and a successful community consultation process has stimulated a good level of involvement in the planning of the works and general community acceptance of the inevitable construction disruption.

It is felt that the simple, responsive management structure established by the City, the cooperation of staff associated City departments, a logical, systematic approach to the planning of the infrastructure and streetscape amenities, and generally well organized, cooperative contractors, have all contributed to the success of this new approach to the urban infrastructure problem.

INSITUFORM - THE NO DIG SOLUTION

Ken Foster
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IGL Canada Limited

INTRODUCTION

Over the past few years the need for rehabilitation of Canada's urban infrastructure has become an accepted fact. The questions that now need to be resolved are how to fund the works and how to undertake the works.

This paper reacts to the second issue and reviews the Insituform process which is a true no-dig solution to the sewer rehabilitation and a cost effective alternative to conventional open cut techniques.

THE PROCESS

The Insituform method at face value is very simple and is familiar now to most engineers. After an appropriate pipeline survey and any necessary preparatory works have been completed a specially coated needled felt tube is manufactured and impregnated with polymer resin. This tube is lowered into the end manhole through an inversion tube, turned inside out and filled with water, usually from a nearby hydrant. The weight of the water in the inversion tube causes the impregnated tube to invert into the pipeline pressing the insitutube against the inside wall of the existing pipe with the specially coated surface becoming a new smooth wall for the pipeline. Once the inversion has been completed, the water used for inversion is circulated through a boiler and the hot water cures the polymer resin to form a new pipe. The new pipe has no joints and is often stronger than the pipe it replaced. The work is completed by cutting the end of the tube and opening any lateral connections. In man-entry pipes the connection will be re-opened using a hand held cutter, whereas in non man-entry pipes remote controlled cutters are used.

Since the insitutube is normally inserted from an existing manhole it doesn't require any excavation resulting in no disturbance to surface features such as roads, trees, fences and buildings or underground utilities which are often in proximity to the sewer.

The process was first used in 1970 and since that time has been used in over 40 countries with over 3000 kilometers having been renovated by this process. Applications vary widely in terms of size, depth, length, pressures and types of effluents handled. It has been used to renovate circular pipes from 100 mm to 2700 mm in diameter with wall thicknesses varying from 3 mm to 50 mm. Insituform is also often used to renovate other shaped sewers such as egg-shaped, box sections, flat tops and manhole chambers.

APPLICATION

The Insituform process was developed to provide a means of rehabilitating existing pipes without excavation. Some typical applications include:

- Strengthening existing structurally deteriorating pipes.
- Eliminating infiltration of ground water in gravity pipelines.
- Reducing maintenance by eliminating root intrusion and deposition of solids.

- Internal corrosion protection of pipelines.
- Prevention of exfiltration of pipeline contents.
- Increasing the flow capacity of existing pipelines by smoothing the interior surface.
- Elimination of leaks in pressure pipes.

These applications include both circular and non circular pipes, square culverts and corrugated metal pipes.

Most of these problems have been encountered and overcome in recent projects in Western Canada.

DESIGN CONSIDERATIONS

The Insituform process is extremely flexible since the materials used can be selected to suit a wide variety of environments. The felt tube is used only as a carrier to place and retain the polymeric material. Three types of polymeric material are available for use in the process, depending on the type of environment to be encountered.

Polyester resins are normally used for municipal sewers, epoxy resins where greater strength is required and for potable water use and vinyl ester resins where greater corrosive and abrasive resistance is required.

The structural design life for insituform is based on a minimum of 50 years calculated in accordance with the Water Research Centre (UK) requirements. The factors which must be considered when specifying an Insitutube for a particular application include the following:

- Diameter (or perimeter) of the pipe.
- Length of continuous run of the pipe for each installation.
- Structural condition of the pipe.
- Pressure or non pressure pipe.
- Depth of pipe.
- Height of water table.
- Type of effluent.

This information is necessary since the insitutube is tailor made to the problem it is to be used to overcome. Based on the information received for each project a detailed design process is followed to determine the thickness of the tube and the type of resin required.

In July 1988 an independent test was conducted at Utah State University to determine the Structural Support that the Insituform system brings to a failing pipe system. This testing clearly demonstrated the significant gain in strength that was obtained when a damaged pipe was repaired using the Insituform process.

Details of this testing program are readily available.

An aspect often overlooked in the design process is the increase in capacity that occurs after a pipe has been insituformed. The much smoother surface of the new pipe, together with the smooth transitions at offsets and protrusions and the elimination of joints, results in increases in flow velocity, often significantly. The smooth surface also improves capacity by reducing deposition. The continuous nature of the new pipe within the old keeps debris moving, increasing flow and reducing maintenance. The insitutube roughness value used in the Manning equation is 0.009 and in the Colebrook - White equation, the equivalent value is 0.6 mm.

SOCIAL AND BUSINESS DISRUPTION

Normally when a renovation project is being proposed the evaluation of the alternatives is based on the actual construction cost of the project. Little if any consideration is given to the costs related to social and business disruption.

A major advantage of the Insituform process is that it is normally a complete "no-dig" solution and as such dramatically reduces the impact on traffic and neighbouring residences and businesses.

On a normal project of one city block length, say 150 to 200 metres, the insituform inversion, curing and re-opening of services would take approximately 24 hours with all access to the sewer being provided by way of existing manholes. If the sewer is located in a street then approximately one lane of traffic will be occupied by the insituform equipment, thereby still allowing traffic to use the street.

When compared to the length of time taken by conventional open cut techniques for a similar job, the reduction in time, disruption and inconvenience is dramatic.

We believe that when a municipality is evaluating alternative methods for undertaking a renovation project this factor should be one of the major considerations.

NEW TECHNOLOGY

Technical centres have been founded in Wellingborough, England and Memphis, USA, to undertake development work to expand and upgrade the technology offered. Currently, three new systems are entering the commercial market and will have an impact on the renovation market. These are light cure, potable water systems and lateral linings.

The light cure system is an interesting alternative to the traditional heat cure. Curing of the polymer is achieved by passing a string of ultra violet lights through the tube after inversion to effect rapid and consistent curing of the resin. The equipment for this system is smaller, less expensive and more versatile. This system will have particular advantage for long inversions, submarine work, or high ambient temperatures where conditions impose tough demands on heat cure apparatus. Where mains electricity is available the curing process is almost silent and the environmental nuisance is minimal. Light cure is over 50% faster than heat cure in many situations. It is anticipated that this system will be available in Canada later in 1989 or early in 1990.

Following years of stringent testing by national authorities in different parts of the world, the insituform process is beginning to receive approval for use in potable water pipelines. Japan was the first country to approve the process followed by Norway and more recently conditional approval was received in the USA. Detailed tests carried out in the United Kingdom have passed the materials as suitable for use in potable water pipelines and final approval is being sought. In order to meet the requirements for potable water the system will use insitutubes with a surlyn nylon coating and epoxy resins.

The application to gain approval in Canada will commence soon and will be based on the detailed testing undertaken in the USA and Europe.

A lateral lining technique has been developed to permit the lining of laterals from within the main in both man entry and non man entry sized sewers. This process will allow a complete insituform structure to be installed without excavation. This lining system is currently under final field testing and will be available to licensees shortly.

Other research is ongoing into improved materials and resins, especially with respect to reinforced insitutubes and improvements in the physical properties of resins.

MANHOLES

It is quite normal when a pipeline is found to be in a state of disrepair that the manholes in the same section are also needing some repair. If this manhole has to be dug-out to repair then some of the advantages of the "no-dig" Insituform process is lost. However, it is now possible to undertake structural concrete repairs to a manhole without excavation. The combination of the Insituform process and this patented manhole repair system allows a complete "no-dig" solution to be undertaken.

This manhole system uses light weight steel panels, engineered to withstand the high loads of wet concrete at considerable depths. Each panel is pre-selected to properly conform to the size and shape of the manhole at each level of the existing manhole while maintaining a minimum annular space of 3" between the existing wall and the new forms. Special reducer sections, designed to accommodate the inward taper of the existing cone section are used. Concrete is poured into the annular space, vibrated and after curing, the form is disassembled and removed. Most manholes are rebuilt in less than one day and, since there is no excavation and no interruption of sewage flows, residents and businesses are often unaware that such work has been undertaken.

Leaks are eliminated at their source joints, cracks and voids are permanently sealed to a condition superior to that obtained when originally installed.

LONGEVITY

A question often asked is "how long will insituform last?" With nearly twenty years of successful installations, there is now a wealth of positive data available to show insituform is likely to be at least as long lasting as any other material in comparable environments.

The very first installation was made in a 1,175 mm x 610 mm egg-shaped in London in 1970, while still in its experimental stages. In 1984, Thames Water Authority removed samples from the pipe for testing. Despite an aggressive environment, caused by hot caustic waste and abrasive solids, the insitupipe was found to be in excellent condition.

Another early installation, constructed in 1972, comprised three 2,450 mm circular pipes at a pumping station in London's east end. In 1982 Thames Water carried out detailed inspections and testing and determined that the pipes were in "first class condition, with no sign of wear or deformation". As a result of this inspection the Authority declared no further testing would be undertaken for another ten years.

These and many other installations have proved that Insituform will last for many years. With the introduction of new unproven techniques which are likely in the next few years, this long product history is important to understand.

ABSTRACTS

The following abstracts represent presentations which were made at the 41st Annual Convention of the Western Canada Water and Wastewater Association. Complete papers were not submitted for inclusion in the Proceedings.

NICE PEOPLE DO COMPLAIN

B. Loree
Citizen Action Centre
City of Edmonton

One of the most challenging tasks in all public sector jobs is dealing with citizens and customers. It can be the most frustrating part of your responsibilities, but it also can be the most satisfying. Communication and information sharing are the keys.

This presentation takes a light-hearted but informative look at the concerns and calls you can expect from the public, and how you can deal with them effectively.

"Nice people do complain" and it is how you respond that sets the image of your company. It can be positive or negative - it is up to you.

UPDATE OF THE OPERATIONS AND MAINTENANCE OF NORTHERN REMOTE COMMUNITY INFRASTRUCTURE SYSTEMS

H.R. Kuehne
Associated Engineering
Edmonton, Alberta

The Alberta Government in conjunction with the Improvement Districts have funded millions of dollars of water and waste water facilities for small, remote communities in northern Alberta. A study was initiated to assess how these facilities are operated and maintained and also to assess the reliability and suitability of those systems.

The findings of the study will be presented and will address the following:

- An overall review and critique of the operations and maintenance procedures currently carried out in northern facilities.
- An assessment of northern facilities and the recommended ways to improve the cost effectiveness of design.
- The socio/cultural setting of the communities as it relates to the appropriate standards of design and levels of operation and maintenance.

In the paper, specific aspects of the study will be presented and will include:

- The use of appropriate technology in the design of remote facilities.
- Feasibility of low-tech systems versus the need of little or no operator involvement.
- The appropriateness of the minimum standards established by Alberta Environment as they apply to remote communities.

- The availability, skills and capabilities of operators in remote communities in relation to recruitment and training.
- Recommended procedures for start up and commissioning of facilities.
- Improved procedures to enhance the operational life of existing and future systems.
- The format of operation and maintenance manual and the monitoring of operation and maintenance procedures.

WATER RATE MAKING PROCESSES AND PRINCIPLES IN THE CITY OF EDMONTON

R. Davis
Environmental Services
Edmonton, AB

Water users should pay their share of the costs of providing water service.

Rate making practice relates the operational and financial costs incurred in providing water service to the average and peak water consumption demands and service requirements.

The rate making process involves the following steps:

(i) Determine objectives, establish test period, accounting convention (cash vs. utility) and allocation method.

(ii) Calculate a revenue requirement (costs to be recovered).

(iii) Functionalize revenue requirement and capital investment (treatment, reservoirs, transmission, distribution, etc.)

(iv) Classify revenue requirement by cost-casual nature: (a) customer costs, (b) commodity costs (average water demands), (c) capacity costs (peak demands), and (d) fire protection.

(v) Allocate total revenue requirements to classes (residential, commercial/industrial, regional, private, fire protection).

(vi) Calculate rates to recover cost allocation to customer class.

Water rate making practices within the City of Edmonton will be presented within this context.

HIGH VOLUME WATER TRANSMISSION LINE INSTALLATION

A. Kuypers
A.K. Construction Ltd.

In 1987 A.K. Construction Ltd. introduced a new piece of equipment to Canada, to trench and lay water transmission lines, forcemains and rural water distribution.

The equipment is a chain trencher built by Dynapac Manufacturing in Germany and was originally designed to install drainage tile in the low lands of Europe.

This trencher is unique because the digging angle always remains at 60 degrees at the various depth required. With the digging angle remaining the same, it allows us to attach a boot to the boom of the trencher which then guides the pipe to the bottom of the trench.

The pipe is protected in the boot which allows us to go through wet areas that would normally cave in or would have to be excavated by backhoe method to a width which would be safe to work in.

On pipe installation normally done by backhoe method, our cost of installation is generally about 25% lower by trencher method.

LIME SLUDGE DEWATERING AT THE RED DEER WATER TREATMENT PLANT

G.A. Irvine and M.F. Adkins
Reid Crowther & Partners Ltd.
R.V. Bayne
City of Red Deer

Red Deer Water Treatment Plant dewaters its lime softenings sludge using a gravity thickener and a vacuum drum-belt filter. This system was designed originally to dewater the sludge produced when treating 41 ML/d of water. However, the dewatering facilities could not satisfy the demand when the treated water production reached only about 32 ML/d. This happened even though these facilities, sized for operation 8 hours per day, were working 24 hours per day.

This paper describes the study conducted to solve the problem described above. A mass balance was calculated for the entire water treatment plant. Settling and filtration bench top tests were conducted. Plant tests were carried out to test the conclusions of the study. The result was to increase the dewatering capacity of the plant to that required at present by simply operating the vacuum filter less than eight hours per day. Other minor modifications should expend the sludge dewatering capabilities to match the water treatment capacity, which is about 107 ML/d.

EXPANSION OF THE CITY OF LETHBRIDGE WASTEWATER TREATMENT PLANT

K.J. Cronin, P.Eng. and M.W. Slezak, P.Eng.
UMA Engineering Ltd.

This paper provides an outline of the main and more interesting features of the expansion of this secondary activated sludge plant, completed in 1988. The City of Lethbridge plant is unusual in that a major waste load, from 6 large food processing plants, is separated out from the domestic system and conveyed to the plant separately. A special industrial agreement provides for these wastes to be treated in a dedicated portion of the plant.

Under the just completed expansion, the domestic portion of the plant was increased from 25,000 cubic metres per day to 40,000 cubic meters per day (average day flows). The industrial portion was modified but remains unchanged at 9,000 cubic metres per day (average day flows).

Some of the main features included in the design and expansion that will be reviewed include:

- The use of fine screens, at 6 mm mesh openings, as part of the pretreatment works. (Most plants use bar screens at an opening of 50 mm.)
- Improved grease and scum handling through the recirculation of warm digested sludge to primary clarifier scum pits.
- The diffused aeration system.
- Anaerobic sludge digester mixing.
- The computerized Distributed Control System.

RECENT DEVELOPMENT IN STORMWATER MANAGEMENT FOR URBAN AREAS

G. Zukovs and T. Field
CH2M Hill Engineering Ltd.
M. Phillips
City of Edmonton

In the course of its recent study into the design and operation of its drainage systems, the City of Edmonton drainage task force has assessed the planning and design procedures used in North America cities. Issues such as level of service, design storms, design of stormwater retention facilities, combined sewer overflows, and methods of financing have been researched and reviewed.

This paper will outline the trends identified during this study. The challenges faced by small and large communities in facilitating urban development - in system design for new areas and rehabilitation of older areas - approaches to dealing with combined sewer overflows, and innovative measures to avoid the costs of increasing the capacities of conveyance systems will be assessed.

IMPACT OF RESIDENTIAL WEEPING TILE FLOWS ON URBAN DRAINAGE SYSTEM DESIGN

T. Field and G. Bontus
CH2M Hill Engineering Ltd.

Weeping tiles have traditionally been installed around residences to depress the localized water table, protect the basement walls from excessive hydraulic forces, and reduce water inflow through cracks, joints, etc. However, the flows from weeping tiles into the urban sewer system have been seriously underestimated in the past.

This paper describes a theoretical basis for calculating the response of weeping tile flow to increases in groundwater level and assesses the results of field tests carried out in various parts of Canada. The paper will also outline the test program planned for Edmonton in 1989 including the methodology used and results obtained from full scale field investigations.

The paper will examine the options available for dealing with residential weeping tile connections to sewer systems and provide recommendations on design flows to be used in design. The impact of these flows on system design will be discussed.

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